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STRESSES IN RAILWAY BRIDGES ON CURVES.

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WITH DISCUSSION.

The complete investigation of the effect produced upon the live load stresses in a bridge by the curvature of the track; by the accompanying super-elevation of the outer rail; and by the eccentricity of the center of the track with reference to the axis of the bridge, has not yet, so far as the author is aware, been published. While the solution of this problem involves only well known methods, it is thought that the general formulas, put in a convenient shape for reference, may be practically useful. That the effect referred to is of sufficient magnitude to demand consideration, even when the degree of curvature of the track is small, is made clear by a comparison of the live load stresses in the members of the inner and outer trusses of the bridge, of which Fig. 1 is the strain sheet for the live load. This bridge, which was recently erected under the supervision of the author, is 187½ feet long, with the track on a 3-degree curve, and was designed by the Keystone Bridge Company. The stresses marked above the chords and diagonals, and on the left of

Fig. 1.

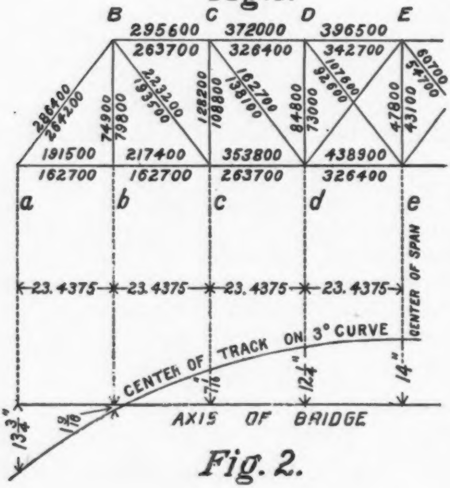
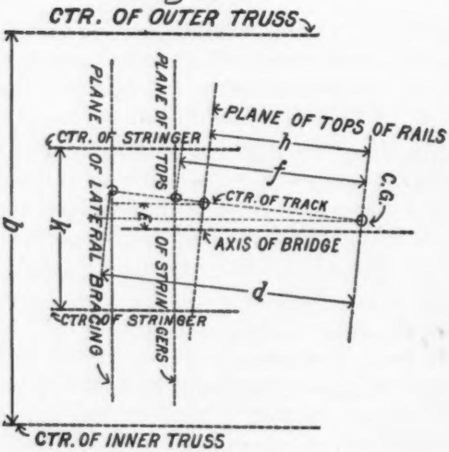


Fig. 2.

Fig. 3.



the verticals are the live load stresses on the outer truss; the other stresses are produced by the live load on the inner truss. The location of the center of the track with reference to the axis of the bridge is shown on an exaggerated scale in Fig. 2. There are two lines of stringers spaced 7½ feet apart on centers, and the axis of the stringers coincides with the axis of the bridge.

First, consider the general case shown in Fig. 3.

Suppose, that for a given position of the train we have at any given panel point, a panel live load "P," for both trusses, and suppose that the track is on a curve of radius "R." In Fig. 3 let the point CG be the center of gravity of the panel live load "P" (this point is taken to be on a normal to the plane of the track through the center of the track, and at a height above the track of the center of gravity of the train); let the super-elevation of the outer rail be s; let the angle of the plane of the track with the horizontal be A, so that sin. $A = \frac{s}{g}$, where $g =$ gauge of track; let E_1 be the eccentricity, or horizontal distance from the axis of the bridge to the center of the track; the positive direction for E_1 being from the axis of the bridge toward the outside of the curve; let h, d and f (all measured in a direction perpendicular to the plane of the track) be the respective distances from CG to the plane of the track, to the plane of the lateral bracing, and to the plane of the tops of the upper flanges of the stringers. Let b be the distance center to center of trusses; k the distance center to center of stringers; and V the velocity of the train in feet per second. Then the centrifugal force of the panel load P is $\frac{PV^2}{32.2R} = PF$; where $F = \frac{V^2}{32.2R}$.

The forces at CG are the centrifugal force PF, considered as acting horizontally, and the panel load P, acting vertically. These two forces constitute the exterior forces acting on the bridge at the panel point considered. These forces must be balanced by the interior stresses induced by them in the trusses, and in the lateral bracing of the bridge.

The centrifugal force will form a couple with the induced shear in the lateral bracing; the moment of this couple being,

$$PFd \cos. A \qquad (1)$$

This couple increases the load on the outer truss by

$$\frac{1}{b}(PFd \cos. A) \qquad (2)$$

and decreases the load on the inner truss by the same amount.

The part of the panel load that is carried by the outer truss is

$$\left(\frac{b}{2} - h \text{ sin. } A + E_1\right)\frac{P}{b} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and the part carried by the inner truss is

$$\left(\frac{b}{2} + h \text{ sin. } A - E_1\right)\frac{P}{b} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

The total outer panel load is found by combining equations (2) and (3) to be

$$\frac{P}{b}\left(\frac{b}{2} - h \text{ sin. } A + E_1 + Fd \text{ cos. } A\right) \ldots\ldots\ldots\ldots\ldots (5)$$

The total inner panel load is found, by combining equations (4) and (2) to be

$$\frac{P}{b}\left(\frac{b}{2} + h \text{ sin. } A - E_1 - Fd \text{ cos. } A\right) \ldots\ldots\ldots\ldots (6)$$

Now, the inner panel load has its greatest value when the second member of equation (6) is zero. This will be the case when the train stands still, for then F will be zero. The maximum inner panel load occurs, therefore, when the train stands still, if impact is neglected; and is found from equation (6), by putting $F = O$, to be

$$\frac{P}{b}\left(\frac{b}{2} + h \text{ sin. } A - E_1\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

The outer panel load, as appears from a consideration of equation (5), has its maximum value when F has its greatest value.

The general formulas are as follows, referring to equations (5) and (7):

Horizontal panel load on lateral bracing

$$P\,F \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (A)$$

Maximum inner panel load

$$\frac{P}{2}\left[1 + 2\left(\frac{h \text{ sin. } A - E_1}{b}\right)\right] \ldots\ldots\ldots\ldots\ldots (B)$$

Maximum outer panel load

$$\frac{P}{2}\left[1 - 2\left(\frac{h \text{ sin. } A - E_1}{b}\right) + \frac{2\,Fd, \text{ cos. } A}{b}\right] \ldots\ldots (D)$$

The application of these formulas to any special case may be simplified by the following considerations:

In equation (A) the factor F depends on the speed of the train, and the curvature of the track. This factor is a constant for all panel loads in any bridge, and needs to be determined but once. The factor $2\left(\frac{h \text{ sin. } A - E_1}{b}\right)$ in equations (B) and (D) depends on the elevation

of the outer rail, and on the eccentricity of the loading at each panel point. This factor must be determined, therefore, for each panel point. The factor $\left(\frac{2\,Fd \cos. A}{b}\right)$ in equation (D) depends on the elevation of the outer rail; on the speed of the train, and on the curvature of the track. This factor is, therefore, a constant for all panel loads, and needs to be determined but once.

Effect on Trusses.—In using these formulas to find the stresses in the truss members, the method of procedure is to find the position of the loading that will produce the maximum stress in the particular member considered, in the same way as if the bridge were on a straight line; then find all the panel loads for this position of the loading; then by means of equations (B) and (D) find the actual panel loads on the inner and outer trusses; then use these panel loads to find the stress in the given member. The value of E_1 in these equations may be taken to be the average value of E_1 in the two panels contiguous to the panel point considered.

As the maximum inner panel load is found for the train standing still, the usual allowance made for impact, if applied to the stresses on the inner truss, will give relatively greater strength for the inner truss than for the outer truss. It is, however, an easy matter to compare the increase of stress due to the specified impact with the decrease of stress due to centrifugal force, given by the last term, of equation (6), if this refinement is thought desirable. The error in adding the impact allowance to the stresses found by using equation (B) will be on the safe side, and it has been the practice of the author to ignore it.

Stringers.—The division of the loading between the stringers in any panel is governed by the same conditions as the division of a panel load between the outer and inner trusses, provided, first, that there are only two track stringers, and that the tops of the stringers are in one horizontal plane; and, second, that the eccentricity of the track with respect to the axis of the stringers is the same throughout the length of each stringer. The first condition is supposed to be complied with in this discussion, as this is usually the case, although some engineers secure the elevation of the outer rail by framing the stringers out of level. The second condition is never realized, but the eccentricity of the track may be taken at the average eccentricity for the stringer considered,

with a close approximation to the truth. Supposing, then, that the tops of the stringers are on the same level, and that E_2 is the average value of the eccentricity of the track with respect to the axis of the stringers for the given panel; the equations expressing the division of the loading between the inner and outer stringers may then be written directly from equations (B) and (D) by substituting k for b, and f for d in those equations. Let L be the total load in the panel, then the load on the inner stringer is,

$$\frac{L}{2}\left[1+\frac{2(h\sin. A-E_2)}{k}\right]\ldots\ldots\ldots\ldots\ldots\ldots(E)$$

and the load on the outer stringer is

$$\frac{L}{2}\left[1-2\left(\frac{h\sin. A-E_2}{k}\right)+\frac{2Ff\cos. A}{k}\right]\ldots\ldots\ldots(F)$$

These equations (E) and (F) also, evidently, give the relation between the maximum stresses in the stringers of any panel for an equal division of the loading between the two stringers, and for the actual division of the loading as determined by the centrifugal force, eccentricity, etc. The actual maximum bending moment and shear for each stringer can be conveniently found, by first finding the maximum bending moment and shear on the stringers when the track is on a straight line, and the stringers are symmetrically placed with reference to the axis of the track; then multiply these stresses by the values of the multiplier of $\frac{L}{2}$ in equations (E) and (F) for each panel, to get the actual stresses.

It is seen from equation (A) that there is also a horizontal shear and bending in the plane of the tops of the stringers, which at any point is equal to the vertical shear or bending at this point, due to the total live load in the panel, multiplied by F. It thus appears that the vertical and horizontal stresses in the stringers can all be found for the case when the track is on a curve by multiplying the corresponding stress in the stringer of a bridge on a straight line by a certain factor.

As the panel length is usually uniform for a given bridge, it is in most cases only necessary to find the maximum bending and shear for one panel, supposing the bridge to be on a straight line, and then find the values of the actual stresses by multiplying by the values of the multiplier of $\frac{L}{2}$ in equations (E) and (F) for each panel. The horizontal stresses in the plane of the tops of the stringers may be resisted by the

stringers acting as independent beams; or by the bracing of the top flanges of the stringers; or by the main laterals, when they are in the plane of the tops of the stringers and are attached to the stringer flanges.

Floor-beams.—When, as in the case of a bridge on a straight line, the center of gravity of the loading, the axis of the stringers, and the center of the floor-beam are all in the same vertical plane, the loading is divided equally between the inner and outer stringers; the greatest live load shear on the floor-beam is equal to the panel load on either the inner or outer truss; and the greatest live load bending moment on the floor-beam is equal to the product of this panel load multiplied by the distance from the center of either truss to the nearer stringer; and if, as is usual, the panel lengths are uniform, these stresses will be the same for all floor-beams.

Now in the case of the track being on a curve, the loading in a panel is not in general equally divided between the stringers, but the effect of eccentricity, centrifugal force, etc., is to change the proportionate division of the loading between the stringers, one stringer having its load increased, and the other stringer having its load decreased by the same amount. The total load on both stringers being, however, the same as when the track is on a straight line, the effect on the floor-beam of this unequal loading of the stringers in a panel is equivalent to that of a couple with vertical forces equal to the increase or decrease of the stringer panel load, and with an arm equal to the distance from center to center of stringers. This couple is balanced by the stresses induced by it in the trusses, the arm of this resisting couple being the distance center to center of trusses.

Two cases may occur, viz.:

First.—Each line of stringers may be in one continuous straight line, in which case the axes of the stringers and bridge are coincident.

Second.—Each line of stringers may be in a broken line, conforming more or less closely with the curve of the track, in which case the axis of the stringers in any given panel does not usually coincide with the axis of the bridge. Each stringer is, of course, parallel to the axis of the bridge.

In either one of these cases, the reaction of each stringer may be found by applying equations (E) and (F); and the stresses in the floor-

beams are then readily determined. The calculations, may, however, be more conveniently made as follows:

First.—Suppose the stringers to be in a straight line, and that the axis of the stringers coincides with the axis of the bridge. If the eccentricity of the track with reference to the axis of the stringers, in the two panels contiguous to the floor-beam considered, be taken at a uniform average value of E_1, then the maximum stringer panel-load at the given floor-beam is found by analogy from equations (B) and (D) to be

$$\text{For the inner stringer, } \frac{P}{2}\left[1+\frac{2\,(h \sin. A - E_1)}{b}\,\frac{b}{k}\right] \ldots\ldots\ldots\ldots \text{(H)}$$

$$\text{For the outer stringer, } \frac{P}{2}\left[1-\left[\frac{2\,(h \sin. A - E_1)}{b}-\frac{2\,Ff \cos. A}{b}\right]\frac{b}{k}\right] \text{(I)}$$

That is, if the train stands still, the inner stringer panel-load is greater than for a bridge on a straight line by

$$\frac{P}{2}\,(h \sin. A - E_1)\,\frac{2}{k} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(J)}$$

and the outer stringer panel-load is less than for a bridge on a straight line by the same amount. Also, if the train moves at the maximum velocity, the outer stringer panel-load is greater than for a bridge on a straight line by

$$\frac{P}{2}\left[\frac{2.Ff.\cos. A}{k}-\frac{2\,(h \sin. A - E_1)}{k}\right] \ldots\ldots\ldots\ldots \text{(K)}$$

and the inner stringer panel-load is less than for a bridge on a straight line by the same amount.

Since the greatest bending in the floor-beam is near the stringer which has the greater panel-load, the values of (J) and (K) show at once near which stringer the greatest bending in the floor-beam occurs. If (J) is greater than (K), the greatest bending is near the inner stringer; if (J) is less than (K), the greatest bending is near the outer stringer. The stringer panel-loads may be found by using (H) and (I); and are needed in finding the bearing value in the floor-beam web of the rivets attaching the stringers. The values of (H) and (I) may evidently be quickly found from (B) and (D).

The other stresses in the floor-beams are conveniently found by first calculating what the stresses would be if the track were straight, and the loading were equally divided between the stringers; and then applying as corrections to these stresses the changes therefrom caused by curvature, etc., as determined from equations (B) and (D). This is readily

done, since the stresses in the floor-beam at any point between a stringer and the nearer truss are proportional to the truss panel-load of that truss.

Second.—Suppose the stringers to be parallel to the axis of the bridge, but not to be in a straight line. In this case, the stresses in the floor-beams can be conveniently found by finding the truss panel-loads by means of equations (B) and (D); and then using these truss panel-loads to determine the stresses in each beam. The reactions at the ends of the stringers may be found by applying equations (E) and (F) to each stringer. In this case the greatest stresses in the floor-beam are not necessarily at the stringer having the greater panel-load.

The average values of E are found in all cases by taking the curve of the track as a polygon whose sides are the chord of the curve in each panel.

Numerical Example.—The application of these formulas to the special case of the bridge shown in Figs. 1, 2 and 3 may, perhaps, make their use better understood.

The data for the bridge are as follows:

$S = 3.1$ inches. $\quad F = \left(\frac{(58\frac{2}{3})^2}{32.2 \times 1910.1}\right) = 0.056$

$g = 56.5$ inches.

$h = 60$ inches. $\quad \text{Sin. A} = \left(\frac{3.1}{56.5}\right) = 0.056$

$d = 78$ inches. $\quad \text{Cos. A} = 0.998.$

$f = 78$ inches. $\quad R = 1910.1$ feet.

$b = 18.5$ feet $= 222$ inches. $\quad k = 7.5$ feet $= 90$ inches.

$V = 58\frac{2}{3}$ feet per second $= 40$ miles per hour.

$$E_1 \text{ for } b = \frac{-13\frac{3}{4} - 1\frac{9}{16} \times 2 + 7\frac{1}{16}}{4} = -2.45 \text{ inches.}$$

$$E_1 \text{ for } c = \frac{-1\frac{9}{16} + 7\frac{1}{16} \times 2 + 12\frac{1}{4}}{4} = +6.2 \text{ inches.}$$

$$E_1 \text{ for } d = \frac{7\frac{1}{16} + 12\frac{1}{4} \times 2 + 14}{4} = +11.4 \text{ inches.}$$

$$E_1 \text{ for } e = \frac{12\frac{1}{4} + 14 \times 2 + 12\frac{1}{4}}{4} = +13\frac{1}{8} \text{ inches.}$$

$$E_2 \text{ for } ab = \frac{-13\frac{3}{4} - 1\frac{9}{16}}{2} = -7.67 \text{ inches.}$$

$$E_2 \text{ for } bc = \frac{-1\frac{9}{16} + 7\frac{1}{16}}{2} = +2.75 \text{ inches.}$$

$$E_2 \text{ for } cd = \frac{7\frac{1}{16} + 12\frac{1}{4}}{2} = +9.66 \text{ inches.}$$

$$E_2 \text{ for } de = \frac{12\frac{1}{4} + 14}{2} = +13\frac{1}{8} \text{ inches.}$$

Equation (A) has the value $P \times 0.056$.

Equation (B) has values as follows:

$$\text{At } b\ldots.\ \frac{P}{2}\left[1 + 2\left(\frac{3.36 + 2.45}{222}\right)\right] = \frac{P}{2}\left[1 + 0.052\right].$$

$$\text{At } c\ldots.\ \frac{P}{2}\left[1 + 2\left(\frac{3.36 - 6.2}{222}\right)\right] = \frac{P}{2}\left[1 - 0.026\right].$$

$$\text{At } d\ldots.\ \frac{P}{2}\left[1 + 2\left(\frac{3.36 - 11.4}{222}\right)\right] = \frac{P}{2}\left[1 - 0.072\right].$$

$$\text{At } e\ldots.\ \frac{P}{2}\left[1 + 2\left(\frac{3.36 - 13.12}{222}\right)\right] = \frac{P}{2}\left[1 - 0.088\right].$$

Equation (D) has values as follows:

$$b\ldots.\ \frac{P}{2}\left[1 - 0.052 + \frac{2 \times 0.056 \times 78 \times 0.998}{222}\right] = \frac{P}{2}\left[1 - 0.013\right].$$

$$\text{At } c\ldots.\ \frac{P}{2}\left[1 + 0.026 + 0.039\right] = \frac{P}{2}\left[1 + 0.065\right].$$

$$\text{At } d\ldots.\ \frac{P}{2}\left[1 + 0.072 + 0.039\right] = \frac{P}{2}\left[1 + 0.111\right].$$

$$\text{At } e\ldots.\ \frac{P}{2}\left[1 + 0.088 + 0.039\right] = \frac{P}{2}\left[1 + 0.127\right].$$

Equation (E) has values as follows:

$$\text{At } b\ldots.\ 1 + \frac{2\,(3.36 + 7.67)}{90} = 1 + 0.245.$$

$$\text{At } c\ldots.\ 1 + \frac{2\,(3.36 - 2.75)}{90} = 1 + 0.014.$$

$$\text{At } d\ldots.\ 1 + \frac{2\,(3.36 - 9.66)}{90} = 1 - 0.140.$$

$$\text{At } e\ldots.\ 1 + \frac{2\,(3.36 - 13.12)}{90} = 1 - 0.217.$$

Equation (F) for b becomes $1 - 0.245 + \dfrac{2 \times 0.056 \times 78 \times 0.998}{90} = 1 - 0.148.$

" " " c " $1 - 0.014 + 0.097 = 1 + 0.083.$

" " " d " $1 + 0.140 + 0.097 = 1 + 0.237.$

" " " e " $1 + 0.217 + 0.097 = 1 + 0.314.$

" (J) " b " $\dfrac{P}{2}\,(3.36 + 2.45)\,\dfrac{2}{90} = + \dfrac{P}{2} \times 0.130.$

" " " c " $\dfrac{P}{2}\,(3.36 - 6.2)\,\dfrac{2}{90} = - \dfrac{P}{2} \times 0.063.$

" " " d " $\dfrac{P}{2}\,(3.36 - 11.4)\,\dfrac{2}{90} = - \dfrac{P}{2} \times 0.179.$

" " " e " $\dfrac{P}{2}\,(3.36 - 13.12)\,\dfrac{2}{90} = - \dfrac{P}{2} \times 0.217.$

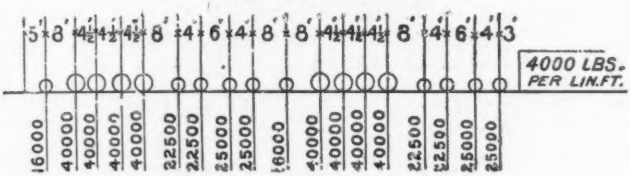
Equation (K) for b becomes $\frac{P}{2}$ $(0.097 - 0.130) = -\frac{P}{2} \times 0.033.$

" " " c " $\frac{P}{2}$ $(0.097 + 0.063) = +\frac{P}{2} \times 0.160.$

" " " d " $\frac{P}{2}$ $(0.097 + 0.179) = +\frac{P}{2} \times 0.276.$

" " " e " $\frac{P}{2}$ $(0.097 + 0.217) = +\frac{P}{2} \times 0.314.$

Diagram of Live Load.



Stringers.

The maximum live load moment on each stringer (supposing the loading to be equally divided between the two stringers), occurs under wheel 4 at $1\frac{1}{8}$ feet from the middle of the stringer; and is 293 000 foot-pounds. Now using the values of equations (E) as factors for the inner stringers, and of equations (F) as factors for the outer stringers, we find the maximum bending moments to be as follows:

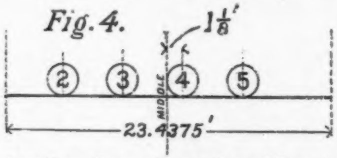


Fig. 4.

TABLE No. 1.

Maximum Moments for Stringers.

	M Maximum for Inner Stringer.	M Maximum for Outer Stringer.
Stringer ab........	293 000 x 1.245 = 364 800 foot-pounds.	293 000 x 0.852 = 249 600 foot-pounds.
" bc........	" x 1.014 = 297 100 "	" x 1.083 = 317 300 "
" cd........	" x 0.860 = 252 000 "	" x 1.237 = 362 400 "
" de........	" x 0.783 = 229 400 "	" x 1.314 = 385 000 "

The maximum horizontal bending moment in the plane of the upper flanges is by equation (A) 293 000 x 2 x 0.056 = 32 800 foot-pounds. The main laterals are, in this bridge, attached to the upper flanges, and thus relieve the stringers from the greater part of this bending. The maximum live load shear at the end of the stringers is (supposing the

loading to be equally divided between the two stringers, and taking wheel (2) at the end of the stringer), equal to 57 900 pounds. Now, using equation (E) as a factor for the inner stringers, and equation (F) for the outer stringers, we find the maximum live load end shears for the stringers to be as follows:

Fig. 5.

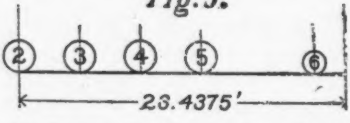


TABLE No. 2.

MAXIMUM END SHEARS FOR STRINGERS.

	S MAXIMUM FOR INNER STRINGER.	S MAXIMUM FOR OUTER STRINGER.
Stringer *ab*	57 900 x 1.245 = 72 100	57 900 x 0.852 = 49 300
" *bc*	" x 1.014 = 58 700	" x 1.083 = 62 700
" *cd*	" x 0.860 = 49 800	" x 1.237 = 71 600
" *de*	" x 0.783 = 45 350	" x 1.314 = 76 100

In this bridge the stringers are riveted to the floor-beam webs, and these end shears in Table No. 2 are used to determine the number of rivets required in the end of the stringer web, and the number of rivets required for single shear in the floor-beam web. The maximum panel load at each stringer is used to determine the number of rivets required for bearing in the connection to the floor-beam web.

The maximum panel live load for one stringer (supposing the loading to be equally divided between the two stringers) occurs with wheel 4 at a panel point, and is 75 900 pounds. Using equation (H) for the inner stringers and equation (I) for the outer stringers, we find the maximum panel live load to be as follows:

Fig. 6.

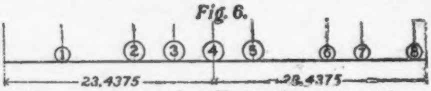


TABLE No. 3.

MAXIMUM PANEL LOADS FOR STRINGERS.

PANEL POINT.	PANEL LOAD.	
	Inner Stringer.	Outer Stringer.
b	75 900 x 1.130 = 85 800	75 900 x 0.967 = 73 400
c	" x 0.937 = 71 100	" x 1.160 = 88 050
d	" x 0.821 = 62 250	" x 1.276 = 96 800
e	" x 0.783 = 59 400	" x 1.314 = 99 700

Floor-beams.

The span of the floor-beams is taken the same as the distance center to center of trusses, namely 18.5 feet. The stringers are 7.5 feet center to center. If the bridge were on a straight line and the loading were equally divided between the stringers, the maximum live load bending moment on the floor-beam would be $75\,900\left(\frac{18.5 - 7.5}{2}\right) = 417\,450$ foot-pounds. In the present case the maximum live load moment occurs at the inner or outer stringer, according to which has the greater panel load. It appears, therefore, from the inspection of Table 3, that the maximum moment occurs at the inner stringer for floor-beam b; and at the outer stringer for the other floor-beams.

The correction for the floor-beam moment and shear is given by (B) for floor-beam b; and by (D) for the other floor-beams.

TABLE No. 4.

Maximum Bending Moments for Floor-beams.

Floor-beam.	M Maximum. Ft. lbs.
b	417 450 x 1.052 = 439 600
c	" x 1.065 = 444 600
d	" x 1.111 = 463 800
e	" x 1.127 = 470 500

TABLE No. 5.

Maximum Shear for Floor-beams.

Floor-beam.	S Maximum.
b	75 900 x 1.052 = 80 000
c	" x 1.065 = 80 800
d	" x 1.111 = 84 300
e	" x 1.127 = 85 500

The longitudinal compression of the upper flange of the floor-beams, and the tensile stresses in the lower chord of the outer truss that are produced by the horizontal component of the centrifugal force, may be computed with sufficient accuracy by using an equivalent uniform panel load. This uniform load is found for the position of the actual loading, giving the maximum stress in the end panel, and gives a uniform panel load of 121 600 pounds for the bridge, or of 60 800 pounds per truss. The

panel centrifugal force due to this panel load is, by equation (A), 121 600 $\times$ 0.056 = 6 810 pounds. The compression on the upper flanges of the floor-beams and the stresses in the laterals due to this horizontal panel load are easily found in the ordinary way, and so are not given here. The tensile stresses in the lower chord of the outer truss due to this horizontal panel load are as follows: The lateral diagonals of this bridge are stiff and form a double lattice system, so that the horizontal shear in each panel may be assumed to divide equally between the two diagonals, giving tension in one and compression in the other. This division of the laterals into two systems of course affects the chord stresses.

TABLE No. 6.

Tension in Lower Chord for Horizontal Force.

Chords.	Stress.
ab	6 810 x 1.75 x 1.267 = 15 100
bc	6 810 (1.75 x 3 — 0.5) 1.267 = 41 000
cd	6 810 (1.75 x 5 — 2) 1.267 = 58 200
de	6 810 (1.75 x 7 — 4.5) 1.267 = 66 900

These stresses are to be added to the tensile stresses in the lower chord of the outer truss that are produced by the vertical loading.

If the equivalent uniform loading of 121 600 pounds per bridge panel be used to find the stresses in the truss members, the work will be very much less than will be required to find the stresses for the actual loading. Both loadings will be used here so that a comparison of their relative accuracy may be made. For a uniform loading the method of procedure is as follows:

Equations (B) and (D) give the values of the panel loads on the two trusses as follows.

TABLE No. 7.

Panel Loads for Uniform Panel Load of 121 600 Pounds.

Truss.	Panel Point.						
	b	*c*	*d*	*e*	*f*	*g*	*h*
Inner	63 960	59 220	56 420	55 440	56 420	59 220	63 960
Outer	60 000	64 750	67 550	68 520	67 550	64 750	60 000

TABLE No. 8.

Reactions at Left Pier for Single Panel Loads.

Truss.	*b*	*c*	*d*	*e*	*f*	*g*	*h*	Total.
Inner...........	55 960	44 410	35 260	27 720	21 160	14 800	8 000	207 310
Outer...........	52 500	48 560	42 220	34 260	25 330	16 190	7 500	226 560

TABLE No. 9.

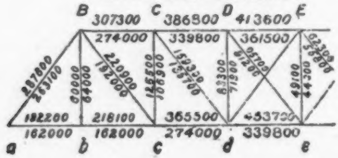
Stresses in Trusses for Uniform Panel Load of 121 600 Pounds.

Member.	Truss	Stress.	From Table No. 6, for Outer Truss.
ab..........	Inner.......	207 300 x 0.78 125 = 162 000	*ab* = 192 200
bc..........	Outer.......	226 600 x " = 177 100	*bc* = 218 100
cd..........	Inner......	350 600 (207 300 x 2 — 64 000) 0.78 125 = 274 000	
BC..........	Outer......	393 200 (226 600 x 2 — 60 000) 0.78 125 = 307 300	*cd* = 365 500
de..........	Inner.......	434 800 (207 300 x 3 — 187 100) 0.78 125 = 339 800	
CD..........	Outer......	495 000 (226 600 x 3 — 184 800) 0.78 125 = 386 800	*de* = 453 700
DE..........	Inner......	462 500 (207 300 x 4 — 366 700) 0.78 125 = 361 500	
	Outer......	529 300 (226 600 x 4 — 377 100) 0.78 125 = 413 600	
aB..........	Inner......	207 300 x 1.269 = 263 100	
	Outer......	226 600 x 1.269 = 287 600	
cB..........	Inner......	151 300 (207 300 — 56 000) 1.269 = 192 000	
	Outer......	174 100 (226 600 — 52 500) 1.269 = 220 900	
dC..........	Inner......	106 900 (207 300 — 100 400) 1.269 = 135 700	
	Outer......	125 500 (226 600 — 101 100) 1.269 = 159 300	
eD..........	Inner......	71 700 (207 300 — 135 600) 1.269 = 91 000	
	Outer......	83 300 (226 600 — 143 300) 1.269 = 105 700	
fE..........	Inner......	44 000 (207 300 — 163 300) 1.269 = 55 800	
	Outer......	49 100 (226 600 — 177 500) 1.269 = 62 300	

These stresses are shown on the strain sheet in Fig. 7.

Fig. 7.

UNIFORM LOADING.



If the actual loading is used the method of procedure is as follows:

First the panel loads are found for each position of the loading that gives a maximum stress in a member. These positions of the loading can be determined graphically from a shear or load diagram with very little labor. The panel loads are then found by the formulas

$$P_n = \frac{M_{n-1} - 2\,M_n + M_{n+1}}{p},$$

where p is the panel length; P_n is the panel load at the panel point; and M_{n-1}, M_n, M_{n+1} are the sums of the moments of all the preceding loads about the $(n\text{-}1)^{th}$, n^{th}, $(n+1)^{th}$ panel points, respectively. This formula was first proposed by Mr. E. Thacher, M. Am. Soc. C. E.

TABLE No. 10.

Panel Loads per Truss for Bridge on Straight Line.

Member.	Position of Load for Maximum Stress.	Panel Load.						
		b	*c*	*d*	*e*	*f*	*g*	*h*
ab and *bc*...........	Wheel 4 at *b*........	75 900	48 600	73 950	56 100	48 100	46 900	46 900
cd and *BC*.........	" 7 " *c*.........	67 000	56 650	62 900	64 750	49 600	49 900	46 900
de and *CD*.........	" 11 " *d*.........	72 700	51 400	68 950	60 900	48 900	46 900	46 900
DE.................	" 13 " *e*.........	30 850	74 000	48 000	75 900	53 500	46 950	46 900
aB.................	" 4 " *b*.........	75 900	48 600	73 950	56 100	48 100	46 900	46 960
cB............	" 4 " *c*.........	17 300	75 900	48 600	73 950	56 100	48 100	46 900
dC............	" 3 " *d*.........	0	8 100	72 600	51 500	68 600	61 100	48 950
eD............	" 3 " *e*.........	0	0	8 100	72 600	51 500	68 600	61 100
fE.................	" 3 " *f*.........	0	0	0	8 100	72 600	51 500	68 600
Equation (B) for inner truss		1.052	0.974	0.928	0.912	0.928	0.974	1.052
Equation (D) for outer truss		0.987	1.065	1.111	1.127	1.111	1.065	0.987

The following table gives the values of the inner and outer panel loads to be used in finding the stresses in the corresponding members.

These values are derived from the above table by multiplying the respective panel loads by the multiplier of $\frac{P}{2}$ in equations (B) and (D).

TABLE No. 11.

Panel Loads on Inner and Outer Trusses for Actual Loading.

Member.	Truss.	Panel Loads.						
		b	*c*	*d*	*e*	*f*	*g*	*h*
ab	Inner	79 800	47 350	68 650	51 150	44 650	45 700	49 350
bc	Outer	74 900	51 750	82 150	63 200	53 450	49 950	46 300
cd	Inner	70 500	55 200	58 350	59 050	46 050	45 700	49 350
BC	Outer	66 100	60 350	69 900	72 950	55 100	49 950	46 300
de	Inner	76 500	50 050	64 000	55 550	45 400	45 700	49 350
CD	Outer	71 750	54 750	76 600	68 650	54 300	49 950	46 300
DE	Inner	32 450	72 100	44 550	69 200	49 650	45 750	49 350
	Outer	30 450	78 800	53 300	85 500	59 450	50 000	46 300
aB	Inner	79 800	47 350	68 650	51 150	44 650	45 700	49 350
	Outer	74 900	51 750	82 150	63 200	53 450	49 950	46 300
cB	Inner	18 200	73 900	45 100	67 450	52 050	46 850	49 350
	Outer	17 100	80 850	54 000	83 350	62 300	51 200	46 300
dC	Inner	0	7 900	67 400	46 950	63 650	59 500	51 500
	Outer	0	8 650	80 650	58 050	76 200	65 100	48 300
eD	Inner	0	0	7 500	66 200	47 800	66 800	64 300
	Outer	0	0	9 000	81 800	57 200	73 050	60 300
fE	Inner	0	0	0	7 400	67 400	50 150	72 150
	Outer	0	0	0	9 150	80 650	54 850	67 700

These panel loads are used to find the stresses in the truss members as given in Tables Nos. 12 and 13.

TABLE No. 12.

Fraction of Panel Load at Left Reaction.

Stress in Member.	Truss.	$\frac{7}{8}b$	$\frac{3}{4}c$	$\frac{5}{8}d$	$\frac{1}{2}e$	$\frac{3}{8}f$	$\frac{1}{4}g$	$\frac{1}{8}h$
ab	Inner	69 820	35 510	42 910	25 580	16 750	11 430	6 170
bc	Outer	65 540	38 810	51 350	31 600	20 050	12 490	5 800
cd	Inner	61 680	41 400	36 470	29 530	17 270	11 430	6 170
Bc	Outer	57 840	45 260	43 690	36 480	20 660	12 490	5 800
de	Inner	66 940	37 540	40 000	27 780	17 020	11 430	6 170
CD	Outer	62 780	41 060	47 880	34 330	20 360	12 490	5 800
DE	Inner	28 400	54 080	27 840	34 600	18 620	11 440	6 170
	Outer	26 650	59 100	33 310	42 750	22 290	12 500	5 800
aB	Inner	69 820	35 510	42 910	25 580	16 750	11 430	6 170
	Outer	65 540	38 810	51 350	31 600	20 050	12 490	5 800
cB	Inner	15 920	55 420	28 190	33 730	19 520	11 710	6 170
	Outer	14 960	60 640	33 750	41 680	23 360	12 800	5 800
dC	Inner	......	5 930	42 120	23 480	23 870	14 900	6 440
	Outer	......	6 490	50 410	29 030	28 570	16 280	6 040
eD	Inner	......	......	4 690	33 100	17 920	16 700	8 040
	Outer	......	......	5 630	40 900	21 450	18 260	7 540
fE	Inner	......	......	......	3 700	25 270	12 540	9 020
	Outer	......	......	......	4 580	30 240	13 710	8 460

TABLE No. 13.

Stresses in Truss Members due to Actual Loading.

Stress in Member.	Truss.	Left Reaction.	Stresses.		From Table No. 6 for Outer Truss.	
ab..........	Inner....	208 170 ×	0.78125 =	162 700	*ab*	191 500
bc..........	Outer....	225 640 ×	0.78125 =	176 350	*bc*	217 400
cd..........	Inner....	(203 950 ×	2 — 70 500) 0.78125 =	263 700	*cd*	354 000
Bc..........	Outer....	(222 200 ×	2 — 66 100) 0.78125 =	295 500		
de..........	Inner....	(206 880 ×	3 — {76 500 × 2 + 50 050}) 0.78125 =	326 400	*de*	438 900
CD..........	Outer....	(224 700 ×	3 — {71 750 × 2 + 54 750}) 0.78125 =	371 950		
DE......	Inner....	(181 150 ×	4 — {32 450 × 3, 72 100 × 2, 44 550}) 0.78125 =	342 700		
	Outer....	(202 400 ×	4 — {30 450 × 3, 78 800 × 2, 53 300 × 1}) 0.78125 =	396 500		
aB.......	Inner....	208 170 ×	1.269 =	264 200		
	Outer....	225 640 ×	1.269 =	286 350		
cB.......	Inner....	(170 660 — 152 460	18 200) × 1.269 =	193 500		
	Outer....	(192 990 — 175 890	17 100) × 1.269 =	223 200		
dC.......	Inner....	(116 740 — 108 840	7 900) × 1.269 =	138 100		
	Outer....	(136 800 — 128 170	8 650) × 1.269 =	162 650		
eD.......	Inner....	(80 450 — 72 950	7 500) × 1.269 =	92 600		
	Outer....	(93 780 — 84 780	9 000) × 1.269 =	107 600		
fE.......	Inner....	(50 530 — 43 130	7 400) × 1.269 =	54 700		
	Outer....	(56 990 — 47 840	9 150) × 1.269 =	60 700		

These stresses are the ones given in the strain sheet in Fig. 1. A comparison of Figs. 1 and 7 will show the difference in the stresses found by these two methods to lie principally in the chords. Of course the floor system should be calculated for the actual loading in every instance.

In the bridge here discussed it will be noticed that the elevation of the outer rail is such that the centrifugal force of the train, *i. e.* (0.056 W*) is exactly balanced by the component of the weight of the train parallel to the track, *i. e.* (W. sin. $A = 0.056\ W$). This elevation of the outer rail is the one that should theoretically exist; and it is sometimes assumed that when this equality of centrifugal force and component of weight parallel to the track exists, the only effect of the centrifugal force will be to produce horizontal stresses in the laterals and chords of the

* This is really (0.056 W cos. A) = (0.056 $W \times$ 0.998) = (0.056 W), appreciably.

loaded chord. The calculations here given, however, show that even when the rail has this elevation, nevertheless the stresses in the web members and unloaded chord are changed quite largely from what they would be for a bridge on a straight line. In many cases the outer rail is not elevated to correspond with the specified velocity; and then, of course, the effect of the curvature on the stresses will be somewhat changed from the case discussed. In fact in this case the elevation of the outer rail was made 2.25 inches, instead of 3.1 inches, so as to have the elevation of the track correspond with that on the two old spans already in place at the ends of this bridge.

Equalized Stresses.

The location of the track with reference to the axis of the bridge is of importance. In the case of a deck bridge this arrangement can usually be made so as to reduce the difference between the stresses on the two trusses to a minimum, which is a desirable thing to accomplish, as the variety of sections is thus reduced. In the case of a through bridge the necessary clearance, as well as the equalization of stresses, is an important factor of the problem. In through spans of moderate length, where the track is straight, it is customary to space the trusses just far enough apart to provide the specified minimum train clearance, as any greater width increases the lengths of the floor-beams, and of all the members of both lateral systems, and also of the transverse dimension of the piers. When the track is on a curve it may either be so located as to secure the minimum specified train clearance at the middle and ends of a through span, thus making the width between the trusses the least possible; or, it may be so located as to equalize as nearly as may be the stresses on both trusses. This latter arrangement will be secured at the expense of an increased amount of material in the floor-beams, laterals and piers, if a given clearance is to be maintained.

As the shifting of the track laterally on the bridge can only effect a transfer of part of the loading from one truss to the other, evidently the total metal in both trusses will not be changed, and there will be no economy secured by so locating the track as to equalize the stresses in the trusses, unless the resulting uniformity of dimensions will enable the contractor to reduce the price per pound.

In the case of the bridge discussed above, the position of the track for an equal clearance between the inner truss and the train at the ends of the span, and between the outer truss and the train at the middle of

the span, is as follows: Let E_a be the negative eccentricity of the center of the track with reference to the axis of the bridge at the ends, and E_b be the positive eccentricity of the center of the track with reference to the axis of the bridge at the middle of the span. The height from the tops of the rails to the bottom of the car is assumed to be 4 feet, and to the top of the car 14 feet. The length of a 34-foot car between truck centers is taken to be 28 feet. The super-elevation of the outer rail is $2\frac{1}{4}$ inches. The horizontal distance from the axis of the bridge to the axis of the floor of the car at the middle of the span is, then,

$$* E_b - 4 \frac{2.25}{4.7} = (E_b - 2) \text{ inches} \ldots\ldots\ldots\ldots\ldots\ldots (\text{M})$$

At the ends of the span the plane of the upper faces of the end posts cuts the plane of the top of the car at about the middle of the end panel, and the versine of a 28-foot chord of a 3-degree curve is about $\frac{5}{8}$ of an inch. Therefore the horizontal distance from the axis of the bridge to the axis of the top of the car at the middle of the end panel is

$$E_a + 14 \frac{2.25}{4.7} + \frac{5}{8} - \frac{13\frac{3}{4} + 1\frac{9}{16}}{2} = \left(E_a - \frac{1}{4}\right) \text{ inches} \ldots\ldots (\text{N})$$

Therefore, for an equal clearance at the ends and middle of the span we have $E_b - 2 = E_a - \frac{1}{4}$; therefore, $E_b - E_a = 1\frac{3}{4}$ inches. Actually $E_b - E_a = \frac{1}{4}$ of an inch, which is nearly correct.

If the super-elevation of the outer rail be taken at 3.1 inches, as used in the calculations, the value of equation (M) becomes

$$E_b - 4 \frac{3.1}{4.7} = (E_b - 2\frac{1}{2}) \text{ inches} \ldots\ldots\ldots\ldots\ldots\ldots (\text{M}_1)$$

And equation (N) becomes

$$E_a + 14 \frac{3.1}{4.7} + \frac{5}{8} - 7\frac{3}{8} = (E_a + 2\frac{1}{4}) \text{ inches} \ldots\ldots\ldots\ldots (\text{N}_1)$$

Therefore, we have for equal clearance $E_b - 2\frac{1}{2} = E_a + 2\frac{1}{4}$; therefore, $E_b - E_a = 4\frac{3}{4}$ inches; $E_a = 11\frac{1}{2}$ inches; and $E_b = 16\frac{1}{4}$ inches. Thus, in this case, the track should be $2\frac{1}{4}$ inches nearer the outer truss than is shown in Fig. 2, for equal clearances at the middle and ends of the span.

Now, suppose that the super-elevation of the outer rail is 3.1 inches, and that $E_a = 11\frac{1}{2}$ inches, and $E_b = 16\frac{1}{4}$ inches. If C inches is the minimum specified clearance between the trusses of a bridge on a

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* The end of the car should be considered, but as it only projects $\frac{5}{16}$ inch beyond the point over truck center, it has been neglected. Eq. (M) should be $(E_b - 2 + \frac{5}{16}) = (E_b - 1\frac{11}{16})$.

straight line, the clearance in this case should be, by equations (M_1) and (N_1)

$$C + 2 \times 13\tfrac{3}{4} = (C + 27\tfrac{1}{2}) \text{ inches} \ldots\ldots\ldots\ldots\ldots\ldots (P)$$

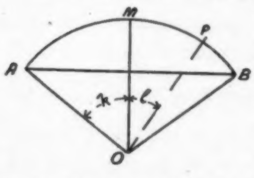
Now, in case the track is so located as to equalize the stress in the two trusses, the track must be moved toward the inner truss $6\tfrac{3}{4}$ * inches nearer than that shown in Fig. 2. In this case, $E_a = 20\tfrac{1}{2}$ inches, and $E_b = 7\tfrac{1}{4}$ inches. The clearance between the trusses must then be, by equation (N_1),

$$C + 2\,(E_a + 2\tfrac{1}{4}) = C + 2 \times 22\tfrac{3}{4} = (C + 45\tfrac{1}{2}) \text{ inches} \ldots\ldots (R)$$

This requires the width between the trusses to be just 18 inches more than is required by equation (P).

This position of the track, that will give equal stresses in the middle chords and end members of both trusses, may be found analytically as follows. Assume the equivalent uniform loading to be distributed uniformly along a curve of the same radius as the center of the track and passing through the center of gravity of the train. The center of mass of this loading, or the center of gravity of the whole train load, is situated at a point at a horizontal distance from the middle of this arc equal to $R\,\frac{S-C}{S}$, where R is the radius of the curve; S is the length of the arc, and C is the length of the chord.

This is readily proved, as follows:† Let AB be a circular arc, which is the locus of a uniform load. Let M be its middle point, and O the center of the circle. Then it is obvious from symmetry that the center of gravity of the loading must lie on the line OM. Take OM as the axis of X. Then, the ordinate of the center



of gravity of an element of load at P is $D_1 = \frac{x.ds}{ds}$, where ds is the length of an elementary arc, and x is the ordinate of P, the center of gravity of the load on the elementary arc. Let l be the angle POM, and R the radius of the circle. Then

$$x = R \cos. l;\ ds = R.dl;\ \text{and } D_1 = R\,\frac{\cos. l\, dl}{dl}.$$

Now if the angle $BOA = 2k$, the integration must be taken from $l = -k$ to $l = k$. Therefore $D_1 = R\,\frac{2R \sin. k}{2Rk} = R\,\frac{C}{S}$; where C is the chord AB; and $S =$ arc AMB.

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* Found by trial.

† This proof is given in Minchin's "Statics."

The distance of the center of gravity of the loading from M is therefore, $D_2 = R\left(1 - \frac{C}{S}\right) = R\frac{S-C}{S}$.

In the present case $R = 1\,910.08$ feet; $C = 187.5$ feet; $S = 187.577$ feet; and $D_2 = \left(1\,910.08\,\frac{187.577 - 187.5}{187.577}\right)12 = 9.44$.

Now the circular arc through the center of gravity of the train is at a horizontal distance from the axis of the track of (60 sin. A) $= (60 \times 0.056) = 3.36$ inches. Therefore the distance of the center of gravity of the train from the middle point of the axis of the track is $D = 9.44 + 3.36 = 12.8$ inches. The centers of gravity of the loads on the arcs AM and MB will also lie on a line parallel to the chord AB, and at 12.8 inches from the point M, because the center of gravity of the load on the whole arc AB will evidently lie on the straight line joining the centers of gravity of the partial loads AM and MB, and this line will necessarily be parallel to AB, by symmetry. If now there were no other forces acting except the vertical load, then the end reactions of both trusses and the stresses in the middle chord members of both trusses would evidently be equal, if the track were so located on the span that the middle point of the axis of the track would have a positive eccentricity of 12.8 inches with reference to the axis of the bridge.

However, the centrifugal force produces a couple whose moment is $W.\ d.\ F.$ cos. $A = 4.35\,W$, where W is the weight of the train. The effect of this couple is to move the line of action of W nearer the outer truss a distance of $\frac{4.35\ W}{W} = 4.35$ inches; hence in order that the resultant action of the whole loading may be at the axis of the bridge, the axis of the track must have a positive eccentricity at its middle point with reference to the axis of the bridge of $(12.8 - 4.35) = 8.45$ inches. This should be the value of E_b if the train conformed to the curvature of the track, but actually the train has the shape of a polygon, and its center of gravity is somewhat farther from M than 12.8 inches; so that E_b is a little too small. This error in the value of E_b is, however, more than compensated by the hypothesis that the eccentricity of the loading in a panel has a value equal to the mean of the eccentricities at the ends of the panel. If the value of E_b be taken at 8.5 inches, and the distance center to center of trusses be made 238 inches, the reactions will be found to be, for the inner truss 217 500 pounds, for the outer truss 215 800 pounds; and the stress on DE for the inner truss to be 380 000 pounds;

and for the outer truss to be 393 500 pounds. This shows that on account of the approximations used in determining the values of E, the value of E_b should be made somewhat less than 8.5 inches, the value found on the hypothesis of the conformity of the shape of the train to the curvature of the track. In this bridge the value of 7¼ inches for E_b appears to be about right, as is seen from the stresses in Table No. 16.

Numerical Example.—For the position of the track given by equation (R) the width center to center of trusses would be 20 feet, and we have E_1 for $b = -9.2$ inches; for $c = -0.55$ inches; for $d = +4.65$ inches; for $e = +6.37$ inches.

Equation (B) for b becomes $\frac{P}{2}\left[1+2\,\frac{3.36+9.2}{240}\right] = \frac{P}{2}\left[1+0.105\right]$.

" " " c " $\frac{P}{2}\left[1+2\,\frac{3.36+0.55}{240}\right] = \frac{P}{2}\left[1+0.033\right]$.

" " " d " $\frac{P}{2}\left[1+2\,\frac{3.36-4.65}{240}\right] = \frac{P}{2}\left[1-0.011\right]$.

" " " e " $\frac{P}{2}\left[1+2\,\frac{3.36-6.37}{240}\right] = \frac{P}{2}\left[1-0.025\right]$.

Equation (D) has values as follows:

$$\text{At } b \ldots \frac{P}{2}\left[1-0.105+0.036\right] = \frac{P}{2}\left[1-0.069\right].$$

$$\text{At } c \ldots \frac{P}{2}\left[1-0.033+0.036\right] = \frac{P}{2}\left[1-0.003\right].$$

$$\text{At } d \ldots \frac{P}{2}\left[1+0.011+0.036\right] = \frac{P}{2}\left[1+0.047\right].$$

$$\text{At } e \ldots \frac{P}{2}\left[1+0.025+0.036\right] = \frac{P}{2}\left[1+0.061\right].$$

Computation for Trusses.—The stresses in the trusses are calculated for a uniform bridge panel load of 121 600 pounds, except bB, for which the maximum truss panel load of 75 900 pounds is used.

TABLE No. 14.

Panel Loads for Uniform Load of 121 600 Pounds per Bridge Panel.

	b	c	d	e	f	g	h
Equation (B) for inner truss....	1.105	1.033	0.989	0.975	0.989	1.033	1.105
Equation (D) for outer truss....	0.931	1.003	1.047	1.061	1.047	1.003	0.931
Inner Panel Load...............	67 180	62 800	60 130	59 280	60 130	62 800	67 180
Outer " "	56 600	60 980	63 660	64 510	63 660	60 980	56 600

TABLE No. 15.

Reactions for Single Panel Loads.

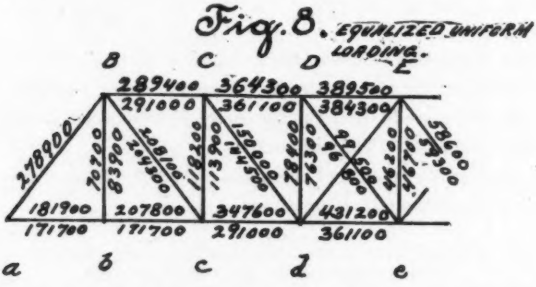
	b	*c*	*d*	*e*	*f*	*g*	*h*	Total.
Inner Truss...........	58 780	47 100	37 580	29 640	22 550	15 700	8 400	219 800
Outer Truss...........	49 530	45 740	39 790	32 260	23 870	15 240	7 080	213 500

TABLE No. 16.

Stresses in Trusses for Uniform Panel Load of 121 600 Pounds.

Member.	Truss.	Stress.		From Table No. 6 for Outer Truss.
ab..........	Inner......	219 800 x 0.78125	171 700	*ab* 181 900
bc..........	Outer......	213 500 x 0.78125	166 800	*bc* 207 800
cd..........	Inner......	372 400 (219 800 x 2 — 67 200) x 0.78125	291 000	
BC..........	Outer......	370 400 (213 500 x 2 — 56 600) x 0.78125	289 400	*cd* 347 600
de..........	Inner......	462 200 (219 800 x 3 — 197 200) x 0.78125	361 100	
CD..........	Outer......	466 300 (213 500 x 3 — 174 200) x 0.78125	364 300	*de* 431 200
DE.......	Inner......	491 900 (219 800 x 4 — 387 300) x 0.78125	384 300	
	Outer......	498 600 (213 500 x 4 — 355 400) x 0.78125	389 500	
aB.......	Inner......	219 800 x 1.269	278 900	
	Outer......	213 500 x 1.269	270 900	
cB.......	Inner......	161 000 (219 800 — 58 800) x 1.269	204 300	
	Outer......	164 000 (213 500 — 49 500) x 1.269	208 100	
dC.......	Inner......	113 900 (219 800 — 105 900) x 1.269	144 500	
	Outer......	118 200 (213 500 — 95 300) x 1.269	150 000	
eD.......	Inner......	76 300 (219 800 — 143 500) x 1.269	96 800	
	Outer......	78 400 (213 500 — 135 100) x 1.269	99 500	
fE.......	Inner......	46 700 (219 800 — 173 100) x 1.269	59 300	
	Outer......	46 200 (213 500 — 167 300) x 1.269	58 600	
bB.......	Inner	75 900 x 1.105	83 900	
	Outer......	75 900 x 0.931	70 700	

These stresses are shown in the strain sheet, Fig. 8.



As will be seen, the stresses on the two trusses are now very nearly equal, except in the case of the lower chords and the suspenders bB; and if the actual loading were used in place of the equivalent uniform loading, the relative equality of stresses would be about the same. Evidently, the outer lower chord must, in any case, be larger than the inner chord, to provide for the additional stress due to the horizontal shear of the centrifugal force. With this exception, all the members of both trusses can be proportioned from the greatest stress on the corresponding members of both trusses, without much waste of material, if, as in this case, the track is so located as to make the average stresses on the inner and outer trusses approximately equal. This will save considerable labor in the calculation of the strain sheet, in the making of working drawings, and in the shop work, on account of the reduction in the variety of sections.

Stringers.—Now, by arranging the stringers properly with reference to the track, the maximum stresses on the inner and outer stringers may be equalized, and it will be possible to make the stringers uniform throughout. To secure this equality of stress in the stringers, equations (E) and (F) must have the value of E_2 so fixed as to make these two equations have the same value in each panel. Suppose the stringers are $7\frac{1}{2}$ feet apart, center to center, then equation (E) becomes

$$1 + \frac{3.36}{45} - \frac{E_2}{45} = 1.075 - \frac{E_2}{45} \ldots\ldots\ldots\ldots\ldots\ldots (E_1)$$

And equation (F) becomes

$$1 - \frac{3.36}{45} + 0.097 + \frac{E_2}{45} = 1.022 + \frac{E_2}{45} \ldots\ldots (F_1)$$

Therefore, for equality of stresses we have

$$1.022 + \frac{E_2}{45} = 1.075 - \frac{E_2}{45}, \text{ and } E_2 = 1\tfrac{3}{16} \text{ inches.}$$

If E_2 is given this value, we have equations (E_1) and (F_1), both equal to 1.049. Therefore, the end shear on each stringer becomes 57 900 x 1.049 = 60 700 pounds, and the maximum bending on each stringer, 293 000 x 1.049 = 307 400 foot-pounds.

The location of track and stringers relatively to the axis of the bridge is given in the following table:

TABLE No. 17.

Eccentricity of Track and Stringers.

At Panel Point	Eccentricity from Axis of Bridge,			Average Eccentricity (from Center of Track) of Axis of Stringers.
	Of Center of Track.	Of Axis of Stringers.		
a	$-13\frac{3}{4} - 6\frac{3}{4} = -20\frac{1}{2} = E_a$	ab	$\left(-20\frac{1}{2} - 8\frac{5}{16}\right)\frac{1}{2} + 1\frac{3}{16} = -13\frac{3}{16}$	$1\frac{3}{16}$ inches
b	$-1\frac{9}{16} - 6\frac{3}{4} = \ldots\ldots -8\frac{5}{16}$	bc	$\left(-8\frac{5}{16} + \frac{5}{16}\right)\frac{1}{2} + 1\frac{3}{16} = \ldots -2\frac{13}{16}$	" "
c	$7\frac{1}{16} - 6\frac{3}{4} = \ldots\ldots \frac{5}{16}$	cd	$\left(\frac{5}{16} + 5\frac{1}{2}\right)\frac{1}{2} + 1\frac{3}{16} = \ldots\ldots +4\frac{1}{8}$	" "
d	$12\frac{1}{4} - 6\frac{3}{4} = \ldots\ldots 5\frac{1}{2}$	de	$\left(5\frac{1}{2} + 7\frac{1}{4}\right)\frac{1}{2} + 1\frac{3}{16} = \ldots +7\frac{9}{16}$	" "
e	$14 - 6\frac{3}{4} = 7\frac{1}{4} = \ldots\ldots E_b$			

The equalization of stresses in the stringers can, evidently, be effected independently of the location of the track with reference to the axis of the bridge. While the advantage of uniformity of dimensions in the stringers may be thus secured; this arrangement has the disadvantage that the number of rivets attaching the stringers to the floor-beam webs is increased, since the stringers in adjacent panels are not in the same line, except at the middle floor-beam.

Floor-beams.—To find the maximum stresses in the floor-beams in this case, where the stresses in trusses and stringers are equalized, take the maximum truss panel live load of 75 900 pounds, and find the panel load on the inner and outer trusses at each point by means of equations (B) and (D). These panel loads give the shears on the floor-beams.

The bending moments on the floor-beams may be found by multiplying these panel loads by the average distance from the end of the floor-beam to the nearer stringer.

TABLE No. 18.

PANEL LOADS FOR UNIFORM LOAD OF 75 900 POUNDS PER TRUSS PANEL.

	b	*c*	*d*	*e*	*f*	*g*	*h*
Inner truss....................	83 860	78 400	75 060	74 000	75 060	78 400	83 860
Outer truss....................	70 660	76 120	79 460	80 530	79 460	76 120	70 660

The length of the floor-beams is taken at 20 feet = 240 inches.

TABLE No. 19.

BENDING MOMENTS ON FLOOR-BEAMS, FOOT-POUNDS.

BEAM.	M AT INNER STRINGER.	M AT OUTER STRINGER.
b..........	$83\,860\left(120-45-8\right)\frac{1}{12}=468\,200$	$70\,600\left(120-45+8\right)\frac{1}{12}=488\,800$
c..........	$78\,400\left(120-45+\frac{5}{8}\right)\frac{1}{12}=494\,100$	$76\,120\left(120-45-\frac{5}{8}\right)\frac{1}{12}=471\,800$
d..........	$75\,060\left(120-45+5\frac{7}{8}\right)\frac{1}{12}=506\,000$	$79\,460\left(120-45-5\frac{7}{8}\right)\frac{1}{12}=457\,800$
e..........	$74\,000\left(120-45+7\frac{9}{16}\right)\frac{1}{12}=509\,100$	$80\,530\left(120-45-7\frac{9}{16}\right)\frac{1}{12}=452\,600$

The maximum bending moments do not differ greatly, and the floor-beams may all be proportioned for a maximum shear of 83 900 pounds, and a maximum bending moment of 506 000 pounds, without involving the waste of much material. The principal items of increased cost, in case the stresses are equalized, are, for this bridge, the additional rivets required in the stringer attachments, and the increased amount of material in the floor-beams and lateral systems due to spacing the trusses 18 inches farther apart. In the bridge, as built, the floor-beams and laterals weigh about 55 000 pounds, and the increase of 18 inches in the width of the bridge will add about 10 per cent., or 5 500 pounds to the weight of metal in the floor-beams and laterals. A more important item, in many cases, would be the cost of an additional 18 inches in the width of the piers. In the bridge discussed above, the piers were already built, and were too narrow to permit the trusses to be spaced farther apart than 18½ feet.

LOCATION OF FLOOR SYSTEM.

The height of the track above the plane of the lateral bracing has an important influence upon the stresses in a bridge on a curve, since any increase of this height increases the lever arm of the centrifugal force, and thus causes a change in the distribution of the loading between the

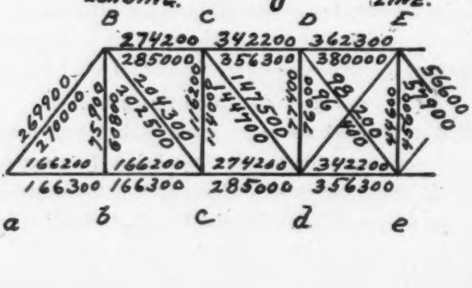
two trusses. In the bridge used in the above calculations, the floor-beams are riveted to the verticals below the lower chord pins, and the lower laterals are just above the tops of the stringers. If the floor-beams were riveted to the verticals above the lower chord pins, and the laterals were under the stringers in the plane of the lower chord centers, the value of d in equation (B) would be about 135 inches, instead of 78 inches, as in the bridge as built.

An examination of the change produced in the stresses by riveting the floor-beams to the verticals above the lower chord-pins will be interesting. Suppose the data are all the same as for the bridge discussed above, except that d is 135 inches.

Then the value of the term $\frac{2Fd \cos. A}{b}$ in equation (B) will be $\frac{2 \times 0.056 \times 135 \times 0.998}{222} = 0.068$. When d is 78 inches, the value of this term is 0.039; so that by making d equal to 135 inches, the panel loads on the outer truss, and, therefore, all the stresses on the outer truss are increased by about 3 per cent. In the case, supposed above, the shears and bending moments in the floor-beams c, d and e would evidently also be increased by 3 per cent. of the stresses that would exist for the bridge on a straight line, in addition to the stresses found for a value of 78 inches for d.

The stresses in the truss members of this bridge for the actual and uniform loading, for the track on a straight line, are given in Fig. 9; the stresses due to the actual loading are above the chords, and diagonals, and on the left of the verticals. The other stresses are due to the equivalent uniform loading of 60 800 pounds per truss panel. This strain sheet is given so as to allow a comparison to be made between the stresses for the track on a curve and on a tangent.

ACTUAL AND UNIFORM LOADING. Fig. 9. STRAIGHT LINE.



GENERAL REMARKS.

Centrifugal Force.—Since the centrifugal force acts radially, the only point at which its line of action is perpendicular to the axis of the bridge is at the middle of the span. At every other point the centrifugal force acts at an angle with the normal to the axis of the bridge. To get the lateral centrifugal force accurately it would be necessary to find at each point the component of the centrifugal force normal to the axis of the bridge. The error resulting from neglecting to consider the inclination of the line of action of the centrifugal force to the normal to the axis of the bridge may be shown to be of no practical consequence in all ordinary cases.

The centrifugal force for a weight p is $p\,\frac{V^2}{32.2\,R} = p\,.\,F$, acting radially. Here $p = w\,.\,ds$; where w is the weight of the train per lineal foot, and ds is the length of an elementary arc.

Therefore the radial centrifugal force for each elementary load is $(F\,.\,w\,.\,ds)$. If the radius makes an angle l with the normal to the axis of the bridge, then the component of the centrifugal force normal to the axis of the bridge is $(F\,.\,w\,.\,\cos.\,l\,.\,ds) = (F\,.\,w\,.\,R\,.\,\cos.\,l\,.\,dl)$, since $ds = R\,.\,dl$.

Let k be half the angle of the curve, then the above expression integrated between $-k$ and $+k$ will give the total centrifugal force normal to the axis of the bridge, for the whole span. This is $(F\,.\,2\,w\,.\,R\,.\,\sin.\,k) = (F\,.\,w\,.\,C.)$, where C is the chord of the curve. The total radial centrifugal force for the whole span is found by integrating $(F\,.\,w\,.\,ds) = (F\,.\,w\,.\,R\,.\,dl)$ between $-k$ and $+k$. This gives $2\,F\,.\,w\,.\,R\,.\,k = F\,.\,w\,.\,S$, where S is the length of the span measured on the center line of the track. Hence the total centrifugal force acting normally to the axis of the bridge is to the total radial centrifugal force as C is to S. This ratio for 300 feet of a 10-degree curve is $\frac{296.6}{300} = 0.99$. The total error is thus seen to be about 1 per cent. of the radial centrifugal force in this extreme case. The greatest discrepancy would, of course, be at the end of the span. For the above case the ratio of the normal to the radial centrifugal force at the end of the span would be cos. 15 degrees $= 0.966$.

Where the conditions would warrant the consideration of this reduction of normal as compared with radial centrifugal force, this can be readily accomplished by multiplying the radial centrifugal force by the

cosine of the angle its line of action makes with the normal to the axis of the bridge. Thus the only change in the general formulas would be the substitution of $F \,.\, \cos.\, l$ for F, where l is the angle that the normal to the axis makes with the radius at the point considered. Usually it need not be considered.

Tabulation of Quantities.—The use of the general formulas in practice can be facilitated by suitable tables of the numerical values of the variables which occur in these formulas. Certain of these variables might properly be designated as variable constants, since they have one value for a given span, but are different for different spans; such quantities are b, d, f, h and k. The values of these quantities are always arbitrarily fixed for a given span; and hence it does not seem possible to construct tables that will cover the different values that may be given to these quantities. Nor does it appear that such tables would prove of much value, as the evaluation of the general formulas for given values of b, d, f, h and k is readily performed. The other variables in these formulas are the sin. A and cos. A; and the quantities F and E. The angle A is usually arbitrarily fixed, by specifying the super-elevation s of the outer rail. The sin. A is then $\frac{s}{g}$, where g is the distance between the points on the rails between which the super-elevation of the outer rail is measured. The cos. A may then be found from a table of sines and cosines. Sometimes the super-elevation s of the outer rail is made such that the component of the load parallel to the plane of the track is just equal to the component of the centrifugal force parallel to the same plane; viz., so that $W.\, \sin.\, A = F.\, W.\, \cos.\, A$, or so that $F = \tan.\, A$. In this case first find F and, taking this for the value of tan. A, find sin. A and cos. A from a table of circular functions. The quantities that require the most labor to evaluate are F and E, and especially the latter. These may be tabulated to advantage.

Values of F.—The value of F is given by the equation $F = \frac{V^2}{32.2\,R}$, where V is the velocity in feet per second, and R is the radius of the curve in feet. The velocity of a railway train is more commonly described as a certain number of miles per hour. Let v be the velocity in miles per hour corresponding to V feet per second; then,

$$V = \frac{5280}{3600}\, v\,;\ \text{and}\ V^2 = \frac{484}{225}\, v^2 .\therefore\ F = \frac{v^2}{32.2\,R} \times \frac{484}{225} = \frac{v^2}{14.97\,R}\ (T).$$

On Plate XCIII the values of $\frac{v^2}{14.97}$ are laid off as ordinates, and the corresponding values of v are the abscissas. To find F for any given

velocity and radius, say 40 miles per hour for a 3-degree curve, take 40 on the axis of abscissas, read the value of the ordinate to the curve of F, in this case 107, and divide this ordinate by $R = 1910.1$. This gives

$$F = \frac{107}{1910.1} = 0.056.$$

In general, find the value of the velocity in miles per hour on the lower row of figures on the axis of abscissas; read from the plate the value of the ordinate to the curve of F at this point; and divide this ordinate by R to find the numerical value of F. The value of F is the centrifugal force per pound of load.

It will be seen that the diagram enables the values of F to be determined for any radius, and for any velocity up to 70 miles per hour. The point for a 3-degree curve and a velocity of 40 miles per hour is marked on the line of F.

Value of E.—In Fig. 10, Plate XCIII, the radius of the circle is R feet. The chord of the arc is C feet. The middle ordinate of the arc is m inches. The angle of the arc is $2\ k$.

Then we have $\sin.\ k = \frac{C}{2\ R}$ .... (V),

$m = R\,(1 - \cos.\ k) \times 12$ inches............................ (W).

On Plate XCIII the values of $12\,000\ (1 - \cos.\ k)$ are laid off as ordinates, and the corresponding values of $\frac{C}{2\ R}$ are the abscissas, thus giving the curve of E.

This diagram is used as follows: For a given chord C, and radius R, find the value of $\frac{C}{2\ R}$; then look for this value of $\frac{C}{2\ R}$ on the axis of abscissas, using the *upper* row of figures; take the ordinate to the curve of E at this point, and multiply it by $\frac{R}{1\,000}$ to get the value of m in inches.

To illustrate the application of the diagram to find the values of E, take as an example the bridge of $187\frac{1}{2}$ feet span, and on a 3-degree curve.

$$\text{Take } \frac{C}{2R} = \frac{187.5}{2 \times 1910.1} = .04908 \text{ for } C_4$$

$$\text{“} \quad = \frac{3}{4} \times .04908 = .03681 \text{ “ } C_3$$

$$\text{“} \quad = \frac{1}{2} \times .04908 = .02454 \text{ “ } C_2$$

$$\text{“} \quad = \frac{1}{4} \times .04908 = .01227 \text{ “ } C_1$$

Now it will be seen that the corresponding ordinates on the diagram are very small, but they can be read to the nearest half tenth without any difficulty. These ordinates are the values of 12 000 (1 — cos. k) for each corresponding value of $\frac{C}{2R}$; and the versines of the arcs corresponding to the chords c_4, c_3, c_2, c_1 may be obtained by multiplying the ordinates in the diagram by $\frac{R}{1000}$, which gives the value of m in equation (W). Having the value of m for each arc, it is an easy matter to find E for each panel point, as will be shown farther on. The values of m found as above are not accurate, because the value of R being so large, the values of $\frac{C}{2R}$ are small, and, as may be seen by referring to the plate, the values of m change more gradually, the smaller $\frac{C}{2R}$ becomes.

This difficulty may be avoided by taking advantage of the peculiar properties of the curve of E, as follows: This curve is approximately a parabola, as may be readily shown. Thus for a given value of the radius and chord, suppose the angle is $2\,k_1$; then $\frac{C_1}{2R} = \sin.\ k_1$ will be the abscissa, and 12 000 (1 — cos. k_1) will be the ordinate. Now take some multiple of sin. k_1 for an abscissa, say $x\,.\,\sin.\ k_1$; then the corresponding chord will be such that $\frac{C_2}{2R} = \sin.\ k_2 = x\,.\,\sin.\ k_1$; and the corresponding ordinate will be 12 000 (1 — cos. k_2). But $1 - \cos.^2\,k_2 = \sin.^2\,k_2 = x^2 \sin.^2\,k_1$, since the sin. k_2 equals x sin. k_1 by hypothesis.

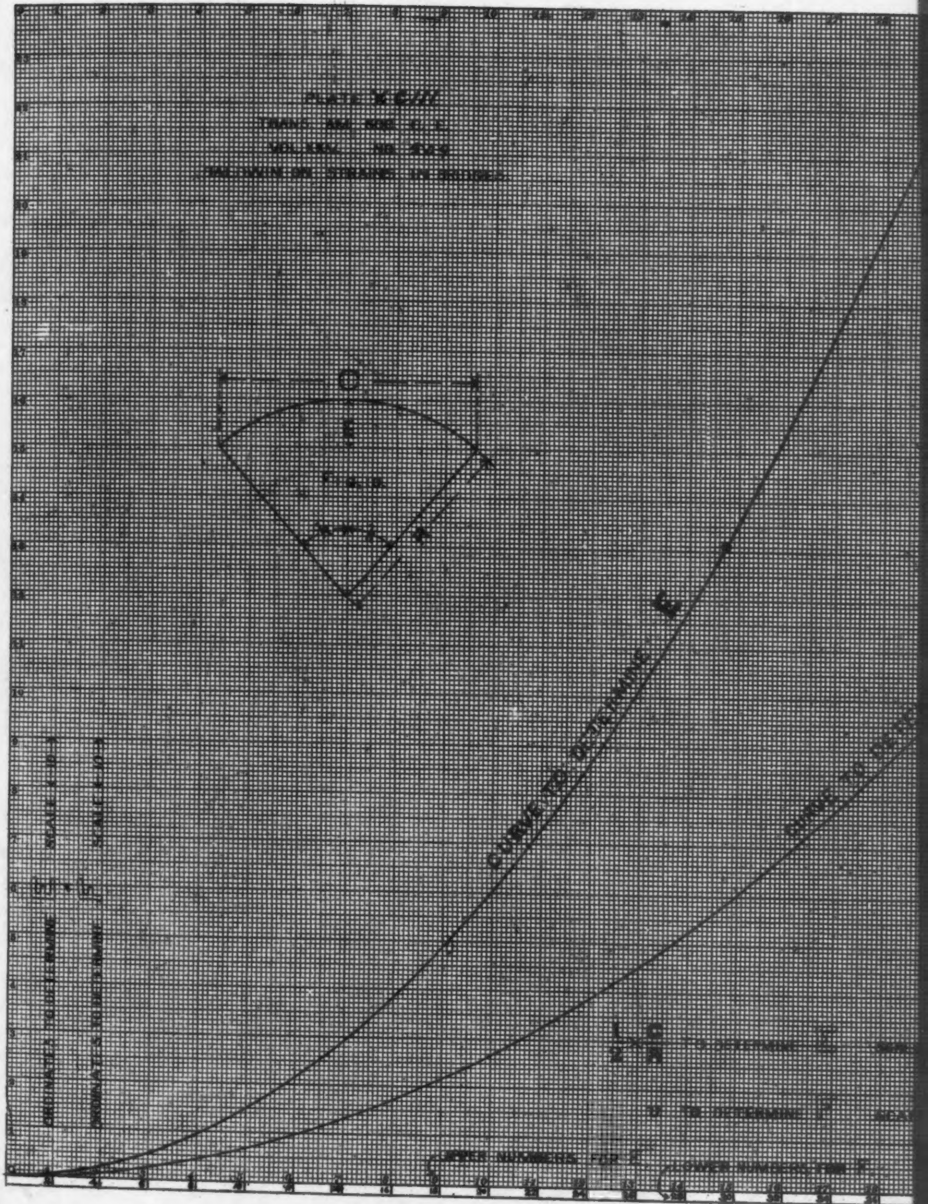
Therefore,

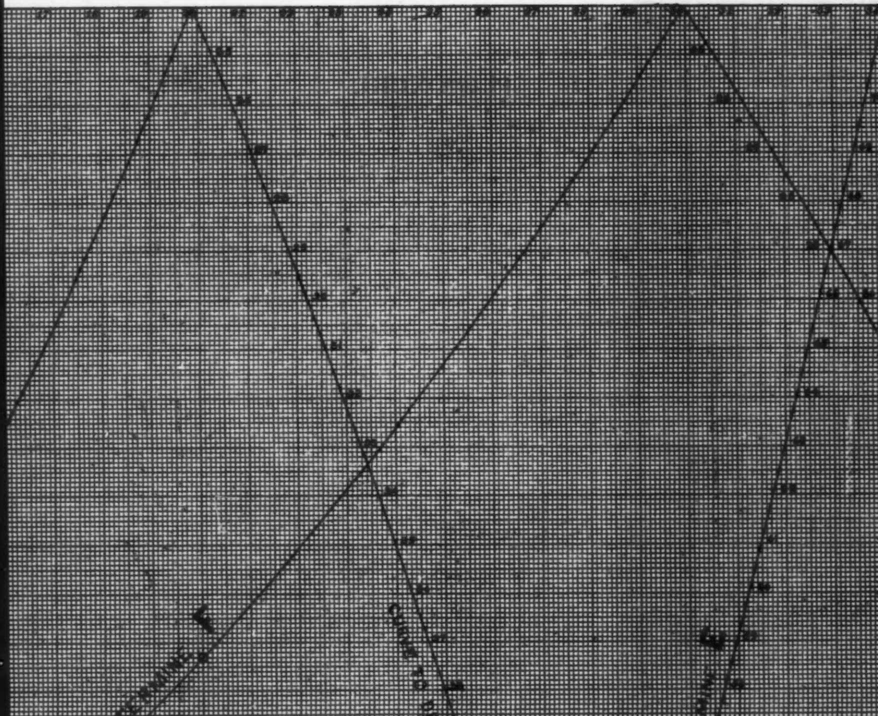
$$1 - \cos.\ k_2 = \frac{x^2 \sin.^2\,k_1}{1 + \cos.\ k_2} = \frac{x^2\,(1 - \cos.^2\,k_1)}{1 + \cos.\ k_2} = x^2\,(1 - \cos.\ k_1)\frac{1 + \cos.\ k_1}{1 + \cos.\ k_2}$$

and hence

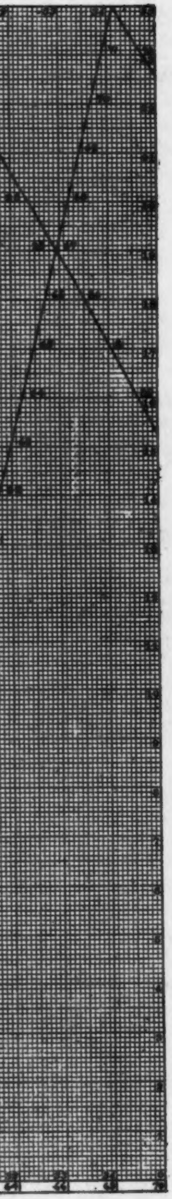
$$12\,000\,(1 - \cos.\ k_2) = 12\,000\,x^2\,(1 - \cos.\ k_1)\,\frac{1 + \cos.\ k_1}{1 + \cos.\ k_2} \ldots\ldots\ldots\ldots\ldots (e)$$

Now, the value of $\frac{1 + \cos.\ k_1}{1 + \cos.\ k_2}$ will be nearly unity when k_2 is small, and also when the ratio between k_1 and k_2 is small. Even when sin. k_2 is as much as 34 times as large as sin. k_1 the value of the above fraction is not much above unity, being 1.0307. The curve is actually drawn for the exact values of 12 000 (1 — cos. k), but equation (e) gives us a useful relation when we consider $\frac{1 + \cos.\ k_1}{1 + \cos.\ k_2} =$ unity, that materially


CURVE TO DETERMINE E
TO DETERMINE



CURVE TO DETERMINE F
CURVE TO DETERMINE
CURVE TO DETERMINE
SCALE—2 × MILES PER HOUR



increases the accuracy and range of the application of the diagram. When $\frac{1 + \cos. k_1}{1 + \cos. k_2}$ is made unity, we have:

$$12\,000\,(1 - \cos. k_2) = 12\,000\,x^2\,(1 - \cos. k_1) = A \,.\, x^2 \ldots\ldots (f)$$

where $A = 12\,000\,(1 - \cos. k_1)$.

Equation (f) enables us to attain minute accuracy in the cases where $\frac{C}{2R}$ is small; for we can, by using this equation, multiply our horizontal scale so as to get our ordinate at a place where the values of $12\,000\,(1 - \cos. k)$ are large, and the percentage of error in reading the diagram is, therefore, small. Thus, suppose we have a value of $\frac{C_1}{2R}$, the corresponding value of $12\,000\,(1 - \cos. k_1)$ being too small to be read accurately from the diagram; then multiply $\frac{C_1}{2R}$ by x, the value of x being such that the ordinate corresponding to $x \,.\, \frac{C_1}{2R} = \frac{C_2}{2R}$ will be large enough to be accurately measured. The ordinate corresponding to $\frac{C_2}{2R}$ is by equation (f), $12\,000\,(1 - \cos. k_2) = 12\,000\,(1 - \cos. k_1) \,.\, x^2$. Therefore read the ordinate on the diagram corresponding to $x\,\frac{C_1}{2R}$, and divide it by x^2, and this latter quotient will be $12\,000\,(1 - \cos. k_1)$. This latter multiplied by $\frac{R}{1\,000}$ gives m. As an illustration take the numerical example above, viz., $R = 1910.1$; $C_1 = 0.01227$; $C_2 = 0.02454$; $C_3 = 0.03681$ and $C_4 = 0.04908$. For C_1 make $x = 12$; for C_2, make $x = 6$; for C_3, make $x = 4$; and for C_4 make $x = 3$. Then

$$x \,.\, \frac{C_1}{2R} = 12 \times 0.01227 = 0.14724$$

$$x \,.\, \frac{C_2}{2R} = 6 \times 0.02454 = 0.14724$$

$$x \,.\, \frac{C_3}{2R} = 4 \times 0.03681 = 0.14724$$

$$x \,.\, \frac{C_4}{2R} = 3 \times 0.04908 = 0.14724$$

Take an abscissa of 0.1472 on the diagram, and read the ordinate corresponding to it, namely, 131. Then we have, for the versines of the arcs,

$$m_1 = \frac{131}{12^2} \times 1.9101 = 1.738 \text{ inches.}$$

$$m_2 = \frac{131}{6^2} \times 1.9101 = 6.950 \quad \text{"}$$

$$m_3 = \frac{131}{4^2} \times 1.9101 = 15.640 \quad \text{"}$$

$$m_4 = \frac{131}{3^2} \times 1.9101 = 27.800 \quad \text{"}$$

Now, by applying equations (*M*) and (*N*) we find the value of E_b for the given case to be 14 inches, say. Then the eccentricities at the panel points are,

at panel point *e*		14	inches.
" " " *d*	(14 — 1.74)	+ 12.26	"
" " " *c*	(14 — 6.95)	+ 7.05	"
" " " *b*	(14 — 15.64)	— 1.64	"
" " " *a*	(14 — 27.8)	— 13.80	"

These correspond closely with the values given in Fig. 2, which latter were found analytically and by a different method. The diagram admits of the accurate determination of the versines of all curves up to an arc of 40 degrees by a simple multiplication, and may be used where it is desired to get the versines to a small fraction of an inch. No interpolation is necessary in using this diagram for *F* and *E*, but these quantities can be evaluated for any value of *v* less than 70 miles an hour, and for any values of *R* and *C*, provided $\frac{C}{2R}$ is less than 0.35; or, what is the same thing, provided that *k* is less than about 20½ degrees.

Wind Pressure.—The general formulas and methods explained above, for finding the stresses due to centrifugal force, may, evidently, be used to find the stresses due to the side pressure of wind on the train; the values of *h*, *d* and *f* being measured from the center of pressure of the wind on the train, instead of from the center of gravity, and the wind pressure being substituted for centrifugal force. With these changes, and with sin. $A = O$; $E = O$; and cos. $A = 1$, equation (*D*) becomes $\frac{w \cdot d}{b}$ for a bridge on a straight line; where $w =$ the wind pressure on train per linear foot.

DISCUSSION.

WILLIAM SCHERZER, M. Am. Soc. C. E.—Mr. Baldwin has presented us with a very complete investigation of the problem of finding the stresses in a railroad bridge when the track is on a curve. The method of finding the position which the track should have relative to the axis of the bridge in order to equalize the stresses in the chords of inside and outside trusses, is novel, and ought to be applied wherever practicable. The local conditions will govern in each individual case as to whether it will be cheaper to increase the width between the trusses, and thereby add a certain amount of metal to the floor beams and lateral bracing, and increase the masonry, or whether it will be cheaper to adopt the minimum width between trusses required for clearance, and have a num-

ber of corresponding truss members of different sections, thereby adding to the cost in the drafting-room and shop. Where the difference in section is slight, the additional shop cost would not be of much consequence, and would be confined mainly to additional care in assembling. Corresponding pins would have the same diameter, pin plates would be of varying thicknesses, but might have the same number of rivets, thus making one templet answer for the corresponding members of both trusses.

In the case of deck truss bridges with both ends supported in the plane of the top chords, the formulas developed by Mr. Baldwin can be applied without any difficulty; but in case one end of the bridge is supported in the plane of the top chord, and the other end in the plane of the bottom chord, with inclined end posts at the latter end, the problem becomes more complicated. Perhaps Mr. Baldwin will find time to extend his investigations so as to include this case. Some years ago, the writer had occasion to calculate the stresses in a skew bridge of the latter type, with trusses of different lengths carrying a track on a 10-degree curve. It was a very difficult problem to solve mathematically, and as the time for investigation was limited, all doubtful stresses were liberally treated with impact, so as to be absolutely certain that the error was on the safe side. A great deal of the ambiguity in stresses might have been avoided if the end supports of these trusses had been kept in the plane of the top chords, making the top chords straight lines.

Viewed from a mathematical standpoint there are some assumptions in Mr. Baldwin's paper which may be open to criticism.

First.—It has been assumed that the centers of gravity of the different wheel loads have the same elevation above the tops of stringers. Is it not probable that these centers of gravity have different elevations, and that the assumption of a mean elevation would materially affect the accuracy of the results?

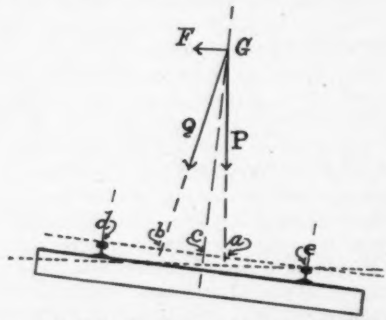
Second.—It has been assumed on page 3 that the centrifugal force acts in a plane parallel to the plane of track, which is incompatible with the assumption that the distance "f" from the center of gravity to the plane of the tops of stringers remains constant. The center of gravity describes a horizontal circle, consequently the resulting centrifugal force will be horizontal, and not inclined, as was assumed. It might be stated, however, that this error will not materially affect the numerical results in the example given.

Third.—The centrifugal forces have all been treated as normal to the axis of the bridge. In cases where the curves are very sharp it would be more correct to treat them as radial forces, and resolve them into components parallel to and normal to the axis of the bridge.

THOMAS H. JOHNSON, M. Am. Soc. C. E.—Mr. Baldwin truly says that the complete investigation of the stresses due to curvature and its accompanying conditions has not been published. Our text books have treated that part of this subject very slightingly; possibly because the

authors regarded curves on bridges as being bad practice, and therefore not to be encouraged; or possibly they thought the elementary principles involved, so simple as not to require special elucidation. But the engineer is controlled to a certain extent by the physical conditions which surround his problem, and to which he must adapt his work. He must therefore expect to build bridges under all possible conditions of curve, grade and skew, singly and in combination; and although the principles involved, in meeting these conditions are few and simple, their application frequently becomes quite complicated; so that ready-made formulas are a grateful relief.

The work of Mr. Baldwin, as presented in this paper, is not speculative but practical, and must be productive of good. In the course of my experience I have had occasion to deal with this problem on several occasions; but approached it in a somewhat different manner from that of Mr. Baldwin. A brief account of the method pursued may not be uninteresting, and may help to throw some side light on the subject.



In the figure let G be the center of gravity of the load at a height $a\ G$ above the top of the rails, which is assumed at 5 feet. Let $d\ e$ = distance from center to center of rail heads, which for the gauge of 4 feet 9½ inches, as used on the lines with which the writer is connected, is 4 feet 11½ inches, so that for all practical purposes the height of the center of gravity may be considered the same as the width between the centers of bearings on the rail heads. This assumption leads to a simplification, in so far as it makes the distance $c\ a$ from the center of the track to the vertical through G equal to the super-elevation of the outer rail.

[Mr. Baldwin uses the gauge of track instead of the center to center width. But the trackman in putting up his track, measures his elevation at the centers of bearing of his level-board, and the wheels, whose effect we are considering, rest on the rail heads, at the same centers. In all formulas either for super-elevation or for the effects of centrifugal force on bridges, this distance between centers should be used and not the

"gauge" in the clear between the rail heads, which has no relation whatever to the problem, either theoretical or practical.]

Let $P =$ the rolling load.

Then $F =$ centrifugal force $= \dfrac{PV^2}{32.2\ R}$ ....(1),

the notation being same as in the paper under discussion. This is the horizontal outward force applied at the rail head, and which must be transmitted as a shear through the cross-ties, stringers and floor beams to the lateral system. Its value in per cent. of P was tabulated for a speed of 50 miles per hour, and for curves from 1 to 12 degrees. This table is given herewith, and shows that for the speed assumed the centrifugal force is 2.9 per cent. of the rolling load for each degree of the curve. In the specifications for bridges—used on these lines—this is made to read 3 per cent. per degree. At 60 miles per hour the rate would become 4.2 per cent., and at 40 miles it would be 1.87 per cent.

The treatment of this horizontal force F as a panel load on the lateral system need not be further discussed here; but we will pass to the consideration of the vertical effects.

Referring to the figure, we have the vertical force P and the horizontal force F, both acting at the same time, and producing the resultant force Q, which cuts the plane of the rail heads, in the point b, and produces an unequal distribution of the load P to the two rail heads. The reactions at d and e will be respectively proportional to the segments $b\ e$ and $b\ d$, into which the distance $d\ e$ is divided by the resultant.

For the condition of train in motion the load on the outer rail only need be considered. This load will be $P^1 = \dfrac{P\ .\ be}{de}$; be being made up of $ab - ac + \frac{1}{2}\ de$, in which $ab = \dfrac{F}{P}\ de$ and $ac =$ super-elevation of outer rail.

The values of P^1 have also been tabulated in per cent. of P, and are given in the fourth column of the table. The load on the inner rail is, of course, the complement of this percentage, and the cross-tie is in the condition of a beam loaded unequally at two fixed points, from which the reactions at each of the stringers may be readily determined. That, in turn, determines two unequal loads at fixed points on the floor beam, from which the reactions at each truss are determined.

The relations of rails to stringers and to the trusses are too various in practice to permit the extension of the table to these members. The eccentricity of the track center, with reference to stringer center and truss center, present too many possible combinations to render such tabulation practicable. But when a problem of this nature is presented it is desirable to determine whether the effects of curvature are of sufficient magnitude to require special provision, without first making the laborious calculations. For this purpose the remaining columns of the table have been prepared, based on all the conditions involved except

the eccentricity of the track center. This eccentricity will be outward at the center of the span and inward at the ends of the span, tending to equalize them, hence this part of the table may be considered as fairly representing the effect of curvature upon the outer truss as a whole, but must not be used to find the effect at each panel point in detail. It serves as a valuable aid to judgment, and shows whether or not the more laborious calculation must be undertaken or may be disregarded; but it should not take the place of such calculations, except, perhaps, in the case of stringers and plate girders spaced 6 feet 6 inches center to center, in which case the column headed with that width is close enough for practice.

For the condition of a train at rest, there being no horizontal component, the reactions at the two rails are determined by the sections *ad* and *ae*, into which the width *de* is divided by the vertical P, which cuts the plane of the rail heads at the point a, distant from c by an amount *ac*, equal to the super-elevation. In this case the inner rail receives the larger share, its load being represented by

$$P'' = \frac{P \cdot ad}{de}$$

which is also given in per cent. of P in the fifth column of the table.

TABLE

Showing the Horizontal and Vertical Effects of Centrifugal Force on Bridges.

Velocity = 50 miles per hour.

Curve.	Radius.	Centrifugal Force.	% of P on		Excess Load on outer Girder expressed in % of ½ P, the normal load.						
			Outer Rail.	Inner Rail.	Widths between Girders.						
Degree.	Feet.	% of P.	Motion.	Rest.	6.5	10	11	12	13	14	15
1	5 730	2.9	51.2	51.6	2.0	1.4	1.2	1.1	1.0	0.9	0.8
2	2 865	5.8	51.3	51.6	6.4	4.2	3.6	3.4	3.0	2.8	2.6
3	1 910	8.7	56.3	52.5	9.6	6.4	5.4	5.0	4.6	4.2	4.0
4	1 433	11.6	58.4	53.3	12.8	8.4	7.2	6.6	6.2	5.8	5.4
5	1 146	14.6	60.4	54.2	16.0	10.6	9.0	8.4	7.8	7.2	6.6
6	955	17.5	62.5	55.0	19.2	12.6	11.0	10.0	9.2	8.6	8.0
7	819	20.4	64.6	55.8	22.4	14.8	12.8	11.6	10.8	10.0	9.4
8	717	23.3	66.7	56.6	25.6	16.8	14.6	13.4	12.4	11.4	10.6
9	637	26.2	68.8	57.5	28.8	19.0	16.4	15.0	13.8	12.8	12.0
10	574	29.1	70.8	58.3	32.0	21.0	18.2	16.6	15.4	14.2	13.4
11	522	32.0	72.9	59.2	35.2	23.2	20.0	18.4	17.0	15.8	14.6
12	478	34.9	75.0	60.0	38.4	25.2	21.8	20.0	18.4	17.2	16.0

E. Thacher, M. Am. Soc. C. E.—I have read Mr. Baldwin's paper with interest and he is entitled to much credit for the thorough and able manner in which he has handled the subject. The formulas given are not difficult of application and will be of service to the profession.

In my own practice in problems of this kind, I have not considered it necessary to go into any great niceties of calculation, but have been satisfied to use such rapid approximate methods as will give results as close to the truth as the data on which they are based, but with formulas ready made, greater exactness can be practiced without great sacrifice of time. To use the actual wheel loads complicates the problem and multiplies the work, and I do not consider it necessary or desirable. To substitute for the wheel loads a uniform load such as will give the maximum shear at the ends of the bridge, gives in the web members essentially the same stresses as from the wheel loads, but slightly in excess toward the center and on the counters, where an excess is usually provided in any event. In the chords at or near the center, the uniform load gives quite a materially greater result, but if the train is considered to precede as well as follow the engines this difference disappears, and the uniform and wheel loads give almost identically the same results; and as this method of loading is not impossible, nor in fact very uncommon, it is questionable if the use of the uniform load is not the best practice.

It appears to be considered by some engineers that the use of a uniform load of any kind is but a rough approximation, but this must arise from a misunderstanding or wrong application of it. For some years past I have been accustomed to use wheel loads in calculation, and to check by uniform loads, which allows of a direct comparison of stresses by the two methods in every estimate. I use the wheel loads not because they are considered better than a uniform load, but simply because the railway companies are satisfied with so doing, and it gives a little cheaper bridge. The use of uniform loads in the calculation of track stringers, floor beams, long suspenders and floor beam hangers, simply allows the use of previously prepared tables; and as the results are the same as from the wheel loads, except slight differences due to interpolation, there is no choice except in time between the two methods. For track stringers the length of span becomes the length of a panel, and the uniform load used gives the same maximum moment as the wheel loads. For floor beams, suspenders and hangers, the length of span becomes the length of two panels.

Equation M for equalizing clearance appears to be taken for a point over the center of the truck, whereas the corner of the car should be taken. Taking a Pullman car 49 feet between centers of trucks and 70 feet in total length, the ordinate for 49 feet $= 1\frac{3}{4}$ inches, and the ordinate at the end of the car $= 2\frac{1}{4}$ inches. $\therefore\ E_b + \frac{1}{4}'' = Ea + \frac{7}{8}''$ or $E_b + Ea = \frac{5}{8}''$ instead of $1\frac{3}{4}$ inches as given.

I cannot quite agree with the author that the height of the track above the plane of the lateral bracing has an important influence upon the stresses in a bridge on a curve. It appears to me that the lever arm of the couple should be measured from the center of gravity of the

train to the center of the bottom chords and not to the center of the lateral system. The laterals, if not placed at the center of the chords, where they belong, exert an excentric stress on the posts, which must be taken care of. The end reactions are a summation of loads at the panel points which govern the stresses in the bridge, and would appear to me to be independent of the position of the lateral bracing. Again, suppose it was possible to place the lateral system at the center of gravity of the train, then the moment of the couple becomes zero, and the centrifugal force has no effect on the stresses, which evidently is incorrect, as the reactions remain unchanged. If the above views are correct, the plane of lateral bracing in Fig. 3, should be marked plane of bottom chords and the notation be made to agree, the formulas remaining unchanged.

Harry B. Seaman, M. Am. Soc. C. E.—The author is in error, in computing a vertical component of the centrifugal force due to the elevation of the outer rail. The centrifugal force acts at the center of gravity of the moving body in the direction of the radius of the curve of motion, and is independent of the shape of the body or its position due to super-elevation.

The action of the centrifugal force noted, as increasing the strains in the outer truss, is interesting and would be even more marked in the case of deck bridges with trusses closer together. With the height of the center of gravity, the distance between trusses, the speed and curvature, each constant, the effect of the centrifugal force on the outer truss is a fixed ratio of the moving load.

Ward Baldwin, M. Am. Soc. C. E. (in reply to the Discussion).—Mr. Scherzer criticises the assumption of a uniform height of the centers of gravity of the different wheel loads. It is true that this is only a rough approximation, but the variations in the heights of these centers for a maximum train load, such as is usually specified for bridge loadings, would not, in my opinion, be great. I am not aware of the existence of any exact data on this point, but here, as is not unusual, the engineer must exercise his judgment, so as to cover probable contingencies, without regard to mathematical precision. For a train of loaded freight cars, equivalent to a uniform load of from 3 000 to 4 000 pounds per lineal foot, hauled by consolidation engines with from 35 000 to 40 000 pounds on a driving axle, the center of gravity of any part of the train would, in my judgment, be between 5 and 6 feet above the tops of the rails. I think 5 feet full small for this height, and believe 5.5 feet would be nearer the truth. As it was my purpose in part to show that the stresses due to centrifugal forces were of sufficient magnitude to deserve attention, I purposely took a minimum value for h, so as to avoid the appearance of exaggeration.

The second criticism offered by Mr. Scherzer is accepted as a practical improvement, and I have modified my paper to accord with the assumption that the center of gravity describes a horizontal circle, and that the

centrifugal force therefore acts horizontally. This is not always the case, however. In fact, in the case of the bridge discussed, the elevation of the outer rail was secured by giving the ties a uniform inclination to the horizon, so that the tops of the cross-ties are all in the same plane, inclined at the angle "A" to the horizon; and the center line of the track at the middle of the span is higher than the center line of the track at the ends of the span by (m. tan. $A = 1.6$ inches), where m is the middle ordinate of the track on the span, or $27\frac{3}{4}$ inches. Thus f is not a constant, but may be assumed so without material error. So, also, while the center of the track is really in the form of an ellipse, inclined to the horizon, still it can be considered a horizontal circle, as Mr. Scherzer suggests, with a close approximation to the truth; and with the advantage of getting simpler equations for the general formula, although not always mathematically correct.

The third criticism offered by Mr. Scherzer suggests a point which I have discussed in the body of my paper. I there show that the normal differs from the radial centrifugal force by only 1 per cent. (of the radial centrifugal force) in the case of a 10-degree curve 300 feet long; and so conclude that the resolution of the centrifugal force is usually not necessary.

Mr. Thacher is correct in saying that the corner of the car should be taken in equation (M); but he has used a Pullman car to figure clearance. I have used a freight car 34 feet long and 28 feet on truck centers, because I have considered it of more importance to so locate the track as to secure equal side clearances for freight than for passenger cars. The ordinate for 28 feet is $\frac{5}{8}$ inch and the ordinate for 34 feet is $\frac{15}{16}$ inch; so that if the truck of the car is at the middle of the span, the value of equation (M) for the end of the car at the floor level would be $(E_b - 2 + \frac{5}{16}) = (E_b - 1\frac{11}{16})$ inches. This gives for equal clearances for the freight car $E_b - 1\frac{11}{16} = E_a - \frac{1}{4}$ $\therefore$ $E_b - E_a = 1\frac{7}{16}$ inches, instead of $1\frac{3}{4}$ inches, as originally given. In the case of the Pullman car, the $2\frac{1}{4}$-inch ordinate at the end of the car is not the projection of the end of the car beyond the truck, but is the middle ordinate of an arc of 70 feet. The projection of the corner of the car beyond the edge of the car at the truck center is $2\frac{1}{4} - 1\frac{3}{4}$ inches $= \frac{1}{2}$ inch. Therefore, for a Pullman car, equations (M) and (N) give $E_b - 1\frac{1}{2} = E_a + \frac{7}{8}$ inches or $E_b - E_a = 2\frac{3}{8}$ inches. The track should, however, in my opinion, be adjusted to give equal clearances for freight cars.

Mr. Thacher has evidently not understood my discussion of the effect of changing the floor system from a position below the bottom chord to a position above the bottom chord. I do not discuss the effect of moving the lateral bracing out of the plane of the chord-pin centers. The two cases were, first, the floor-beams attached below the bottom chord; second, the floor-beams attached to posts above the bottom chord. In both cases the laterals were supposed to be in the plane of the bottom chord-pin centers. I have modified my language in this part of my

paper so as to make this point clearer. Of course, the laterals ought to always be in the plane of the lower chord-pin centers. If, however, it were possible to place the lateral system at the center of gravity of the train, as Mr. Thacher suggests, then, as he says, the moment of the couple becomes zero; but the horizontal action of the centrifugal force is not annulled, and it would produce its full effect upon the laterals, and put a cross strain on the posts unless there were a longitudinal chord at the same level as the attachments of the laterals to the posts. The horizontal reactions due to the centrifugal force are always the same, but the vertical reactions depend entirely upon the relative positions of the center of gravity of the load and the plane of the lateral bracing, which latter ought to always coincide with the plane of the bottom chord-pin centers in a through span.

If the center of gravity is above the lateral bracing the vertical reaction of the outer truss is increased; if the center of gravity is in the plane of the lateral bracing, the vertical reaction of neither truss is affected by the centrifugal force; if the center of gravity is below the lateral bracing, the vertical reaction of the inner truss is increased. I think Mr. Thacher will agree with my views when he understands what they are.

The measurement of the super-elevation of the outer rail between the gauge lines, to which Mr. Johnson takes exception, gives the same value for A that would be obtained if the value of g were made the distance from center to center of rail heads; for the ratio $\frac{s}{g}$ is the quantity that appears in the equations, and evidently s and g increase in the same proportion as long as A is constant. While it may be in accordance with practice to refer s and g to the centers of the rail heads, the numerical results will not be affected by such change in the values of s and g, since $\frac{s}{g}$ remains the same.

Mr. Seaman is mistaken in supposing that I consider the centrifugal force to act in any other way than radially. The center of gravity of the train does not always move in a horizontal circle, as Mr. Seaman assumes. This is only true when the axis of the stringers has the same curve as the track. In the bridge, used as an example in this paper, the tops of the cross-ties are in a plane inclined at an angle A to the horizontal; the center line of the track is laid out with a horizontal radius, and therefore lies on the surface of a vertical cylinder whose radius is the nominal radius of the track. The plane of the cross-ties, being inclined to the axis of the cylinder, cuts the surface of the cylinder in an ellipse; and therefore the actual path of the center of gravity of the train is an ellipse in a plane parallel to the plane of the cross-ties; the radius of the path of the center of gravity at any point is the radius of curvature of this ellipse at this point; and the radial centrifugal force is parallel to the plane of the cross-ties, just as I took it to be. Practically, when A is small, this refinement can be ignored, and the radius be assumed to be horizontal.

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"DIMENSION STONE QUARRYING.—THE BLASTING PROCESS."

By WILLIAM L. SAUNDERS, M. Am. Soc. C. E.

WITH DISCUSSION.

There are doubtless many persons, some of them, perhaps, not without a knowledge of quarrying, who have read extracts from the Census Reports, and have noted frequent reference to a system of blasting known as the Knox system. This system is a recent invention, no mention was made of it in the tenth census, and no description has yet been given of it in any publications on quarrying. The first work done by this method was in 1885, and at the close of that year two quarries had adopted it. In 1886 it was used in twenty quarries. In 1887 in forty-four, in 1888 in upward of one hundred, and at the present time about three hundred quarries have adopted it. Its purpose is to release dimension stone from its place in the bed, by so directing an explosive force that it is made to cleave the rock in a prescribed line and without injury. The system is also used for breaking up detached blocks of stone into smaller sizes.

Quarrymen have, ever since the introduction of blasting, tried to direct the blast so as to save stock. Holes drilled by hand are seldom round. The shape of the bit and the irregular rotation while drilling, usually produce a hole with a triangular section, the walls of the hole taking the shape shown in Fig. 1. It was observed, many years ago, that when a blast was fired in a hand-drilled hole, the rock usually broke in three directions radiating from the points of the triangle in the hole. This led quarrymen to look for a means by which the hole might be shaped in accordance with a prescribed direction of cleavage.

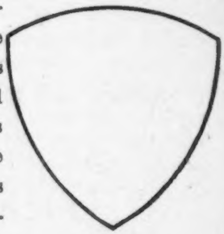


FIG. 1.

The oldest sandstone quarries in America are those at Portland, Conn. It was from these quarries that great quantities of brownstone were shipped for buildings in New York. The typical "brownstone front" is all built of Portland stone. As the Portland quarries were carried to great depths the thickness of bed increased, as it usually does in quarries. With beds from 10 to 20 feet deep, all of solid and valuable brownstone, it became a matter of importance that some device should be applied which would shear the stone from its bed without loss of stock and without the necessity of making artificial beds at short distances. A system was adopted and used successfully for a number of years which comprised the drilling of deep holes from 10 to 12 inches in diameter, and charging them with explosives placed in a canister of peculiar shape. The drilling of this hole is so interesting as to warrant a passing notice. The system was similar to that followed with the old-fashioned drop drill. The weight of the bit was the force which struck the blow, and this bit was simply raised or lowered by a crank turned by two men at the wheel. The bit resembled a broad axe in shape, in that it was extremely broad, tapering to a sharp point and convex along the edge.

FIG. 2.

Fig. 2 illustrates in section, one of the Portland drills, and a drill-hole with the canister containing the explosive in place. This canister was lune-shaped, that is, its section was bounded by two minor segments of a circle. The canister was made of two curved pieces of sheet tin with soldered edges, cloth or paper being used at the ends. It was

surrounded with sand or earth, so that the effect of the blast was practically the same as though the hole were drilled in the shape of the canister. In other words, the old Portland system was to drill a large, round hole, put in a canister, and then fill up a good part of the hole. Were it possible to drill the hole in the shape of the canister, it would obviously save a good deal of work which had to be undone. The Portland system was, therefore, an extravagant one, but the results accomplished were such as to fully warrant its use. Straight and true breaks were made, following the line of the longer axis of the canister section. See Fig. 3.

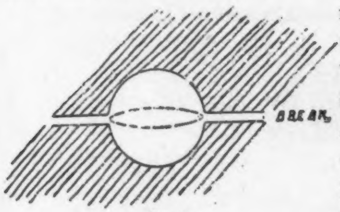


FIG. 3.

It was found that with the old Portland canister two breaks might be made at right angles by a single blast, when using a canister shaped like a square prism. In some of the larger blasts, where blocks weighing in the neighborhood of 2 000 tons were sheared on the bed, two holes as deep as 20 feet were drilled close together, the core between the holes was then chipped out and large canisters measuring 2 feet across from edge to edge were used.

Another of the older systems of blasting is that known as Lewising, Fig. 4. A Lewis hole is made by drilling two or three holes close together and parallel with each other, the partitions between the holes being broken down by using what is known as a broach. Thus a wide hole or groove is formed in which powder is inserted, either by ramming it directly in the hole, or by putting it in a canister, shaped somewhat like the Lewis hole trench. A complex Lewis hole is the combination of three drill holes, while a compound Lewis hole contains four holes. Lewising is confined almost entirely to granite. In some cases a series of Lewis holes is put in along the bench at distances of 10 and 25 feet apart, or even greater, each Lewis hole being situated equidistant from the face of the bench. The holes are blasted simultaneously by the electric battery.

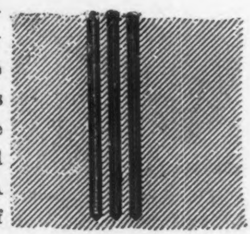


FIG. 4.

Another system used to a limited extent, and by no means to be commended, is one involving the use of inverted plugs and feathers. The hole, which in this case is an ordinary one, is first charged with powder, then the plugs and feathers are inserted as a sort of tamping; the effect

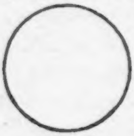


FIG. 5.


FIG. 7.


FIG. 6.

of the blast being to drive the plugs between the feathers and to split the rock by the usual plug and feather process. This is rather a violent way of using this process, and it frequently results in irregular breaks and in damage to the rock at the top of the hole.

It is thus seen that the "state of the art" has been progressive, though it was imperfect. Mr. Sperr, in his reference to this subject made in the report of the Tenth Census, says: "The influence of the shape of the drill-hole upon the effects of the blast does not seem to be generally known, and a great waste of material necessarily follows." This was written but a few years before the introduction of the new system, and it is doubtless true that attention was thus widely directed to the conspicuous waste due to a lack of knowledge of the influence of the shape of a drill-hole on the effect of a blast. The system developed by Mr. Knox practically does all and more than was done by the old Portland system, and it does it at far less expense. It can best be described by illustrations.

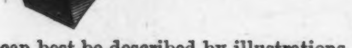


Fig. 8
Reamer.

Fig. 5 is a round hole drilled either by hand or otherwise, preferably otherwise, because an important point is to get it round. Fig. 6 is the improved form of hole, and this is made by inserting a reamer, Fig. 7, into the hole in the line of the proposed fracture, thus cutting two V-shaped grooves into the walls of the hole. The reamers are further shown in Fig. 8. The blacksmith tools for dressing the reamers are shown in Fig. 9. The usual method of charging and tamping a hole in using the new system is shown in Fig 10. The charge of powder is shown at *C*, the air-space at *B*, and the tamping at *A*.

Fig. 11 is a special hole for use in thin beds of rock. The charge of powder is shown at *D*, the rod to sustain tamping at *B*, air-space at *CC*, and tamping at *A*.

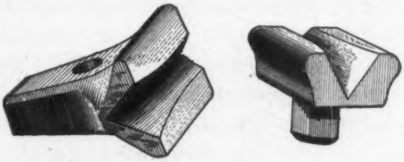


FIG. 9.

Let us assume that we have a bluestone quarry in which we may illustrate the simplest application of the new system. The sheet of stone which we wish to shear from place has a bed running horizontally at a depth of, say, 10 feet. One face is in front, and a natural seam divides the bed at each end at the walls of the quarry. We now have a block of stone, say 50 feet long, with all its faces free, except one—that opposite and corresponding with the bench. One or more of the specially formed

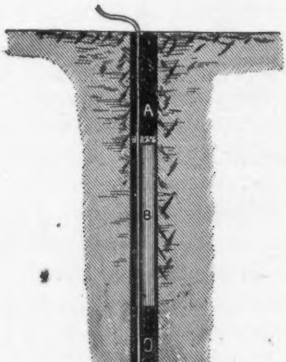


FIG. 10.

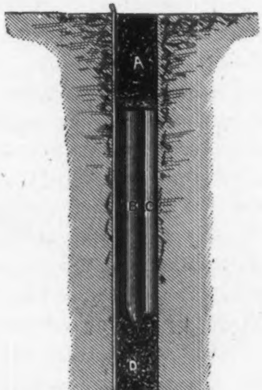


FIG. 11.

holes are put in of such depth and distance apart, and from the bench, as may be regulated by the thickness, strength and character of the rock. No man is so good a judge of this as the quarry foreman who has used and studied the effect of this system in his quarry. Great care should be taken to drill the holes round and in a straight line. In sandstone of medium hardness these holes may be situated 10, 12, or 15 feet apart.

If the bed is a tight one, that is where it is not entirely free at the bottom, the hole should be run entirely through the sheet and to the bed; but with an open free bed, holes of less depth will suffice.

The reamer should now be used and driven by hand. Several devices have been applied to rock drills for reaming the hole by machinery while drilling; that is, efforts have been made to combine the drill and the reamer. Such efforts have met with only partial success. The perfect alignment of the reamer is so important that where power is used this point is apt to be neglected. It is also a well known fact that the process of reaming by hand is not a difficult or a slow one. The drilling of the hole requires the greatest amount of work. After this has been done it is a simple matter to cut the V-shaped grooves. The reamer should be applied at the center of the hole, that is, the grooves should be cut on the axis or full diameter of the hole. The gauge of the reamer should be at least 1½ times the diameter of the hole. While driving the reamer great care should be taken that it does not twist, as the break may thereby be deflected; and the reaming must be done also to the full depth of the hole.

The hole is now ready for charging. The powder should be a low grade of explosive. Dynamite is not suitable, and Black powder, Judson powder, or other explosives which act slowly are preferable. No definite rule can be laid down as to the amount of powder to be used, but it is well to bear in mind that it should be as small as possible. As a matter of fact, very little powder is required in most rocks. Hard and fine grained stone requires less powder than soft stone. This is based on the same principle of philosophy that granite will split by the plug and feather process with holes of less diameter and depth and with less expenditure of force than sandstone. Mr. Knox tells of a case which came under his observation, where a block of granite "more than 400 tons weight, split clear in two with 13 ounces of FF powder." He compares this with a block of sandstone of less than 100 tons weight "barely started with 2½ pounds of the same grade of powder, and requiring a second shot to remove it."

It is obvious that enough powder must be inserted in the hole to produce a force sufficient to move the entire mass of rock on its bed. In some kinds of stone, notably sandstone, the material is so soft that it will break when acted upon by the force necessary to shear the block. In cases of this kind a number of holes should be drilled and fired

simultaneously by the electric battery. In such work it is usual to put in the holes only 4 or 5 feet apart. The powder must, of course, be provided with a fuse or preferably a fulminating cap. It is well to insert the cap at or near the bottom of the cartridge, as shown in Figs. 10 and 11.

After the charge the usual thing to do is to insert tamping, but in the improved form of hole the tamping should not be put directly upon the powder, but an air space should be left, as shown at *B*, Fig. 10. The best way to tamp, leaving an air space, is first to insert a wad which may be of oakum, hay, grass, paper or other similar material. The tamping should be placed from 6 to 12 inches below the mouth of the hole. In some kinds of stone a less distance will suffice, and it is well to bear in mind that as much air space as practicable should intervene between the explosive and the tamping. Care should be observed in tamping not to destroy the wires which connect with the explosive, but the tamping should be made secure, so that it will not blow out. The hole is now ready for blasting. If several holes are used on a line they should be connected in series, Fig. 12, and blasted simultaneously by electricity. The effect of the blast is to make a vertical seam connecting the holes, and the entire mass of rock is sheared several inches or more.

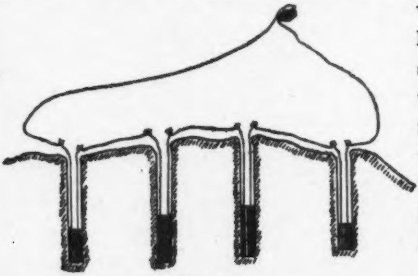


FIG. 12.

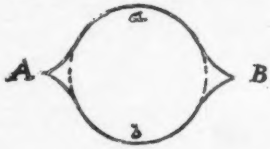


FIG. 13.

The philosophy of this new method of blasting is simple, though a matter of some dispute. The following explanation has been given. See Fig. 13.

"The two surfaces, *a* and *b*, being of equal area, must receive an equal amount of the force generated by the conversion of the explosive into gas. These surfaces being smooth and presenting no angle between the points *A* and *B*, furnish no starting point for a fracture, but at these points the lines meet at a sharp angle including between

them a wedge-shaped space. The gas acting equally in all directions from the center is forced into the two opposite wedge-shaped spaces, and the impact being instantaneous, the effect is precisely similar to that of two solid wedges driven from the center by a force equally prompt and energetic. All rocks possess the property of elasticity in a greater or less degree, and this principle being excited to the point of rupture at the points *A* and *B*, the gas enters the crack and the rock is split in a straight line, simply because under the circumstances it cannot split in any other way."

Another theory has been stated as follows: "A round hole forms on all sides a perfect arch, and, if the rock be sound, it is equally strong in all directions. The making of the grooves at opposite sides of the hole breaks the arch at these sides, thus producing two weak sides and two strong sides at right angles with each other. Force being applied within the hole for the purpose of breaking the rock, naturally exerts itself in the lines of weakness which have been produced by destroying the arches at *A* and *B*, and they, being exactly opposite to each other, the result is that the rock is fractured in a straight line; the gas generated by the explosion acting in these lines in the same manner as a line of wedges would, if applied from the outer side of a rock, with this difference, viz.: Wedges applied from the outer side of a rock are driven inward toward the point of greatest resistance in the rock, whilst the gas being confined within the rock at its strongest part and operating toward the outside, or weaker part, will naturally take that direction which will most quickly relieve the pressure, and that is a direct line to the surface."

The effect of the new system being practically the same as that of the old Portland system, or that of Lewising, it is natural that we should look into these systems in our efforts to explain why the rock breaks in a prescribed line. Good and true breaks have been made by Lewising, yet there is no V-shaped groove. Equally clear and efficient is the record of the old Portland canister, yet here too there are no V-shaped grooves. It might be argued that the Portland canister being imbedded in sand or other non-elastic material, forms a "wedge-shaped space," and here too, "The gas is forced into the two opposite 'wedge-shaped spaces,' and the impact being instantaneous, the effect is precisely similar to that of two solid wedges driven from the center by a force equally prompt and energetic."

But we are met by the evidence of the Lewis-hole, where there is no "wedge-shaped space." While it is doubtless true that the "wedge-

shaped space" is an influence which assists the break, and that the breaking of the arch of the hole by the groove renders equally great assistance in that it produces a weak point to start the break, yet the main cause in the new form of blast which acts to direct the break is, that the lines of force are exerted against the surfaces of the hole, *A b B* and *A a B*, to a greater extent than upon the surfaces *a A b* and *a B b*. In other words, there is a greater area of pressure acting toward *a* and *b*, and this naturally tends to produce a separation at *A* and *B*. The tension is precisely the same as that produced by a line of plugs and feathers, or by a series of wedges driven in a trench.

Let us assume that an effort was made to split by the new form of holes without an explosion, but by using hydraulic or other pressure within the holes. It is obvious that were we able to get the pressure high enough, the break would be made in the same way as though it were blasted, and the very purpose of the air cushion in the new system is to prevent the shock of the blast from having a bad effect upon the rock, and to confine the gas and air at a high pressure in the hole. The suddenness of the explosion is relieved by an expansion and compression of the air cushion, and it is the recoil of the energy after such compression which exercises the greatest force in breaking the rock. Notwithstanding the cushion there is some shock, and this so far assists the break as to enable the operator to use but little explosive. Were the force exerted through hydraulic pressure, it would be advisable to produce a shock, just as a quarryman will strike a block of granite with a heavy sledge in order to start the break, while his plugs and feathers are under strain.

The new form of hole is, therefore, almost identical in principle with the old Portland canister, except that it has the great advantage of the shaped groove in the rock, which serves as a starting point for the break. It is also more economical than the Portland canister in that it requires less drilling and the waste of stone is less. It is, therefore, not only more economical than any other system of blasting, but it is more certain, and in this respect it is vastly superior to any other blasting system, because stone is valuable, and anything which adds to the certainty of the break also adds to the profit of the quarryman.

It is doubtless true that, notwithstanding the greater area of pressure in the new form of hole, the break would not invariably follow the prescribed line but for the V-shaped groove which virtually starts it. A

bolt, when strained, will break in the thread whether this be the smallest section or not, because the thread is a starting point for the break. A rod of glass is broken with a slight jar provided a groove has been filed in its surface. Numerous other instances might be cited to prove the value of the groove. Elasticity in rock is a pronounced feature, which varies to a greater or less extent; but it is always more or less present. A sandstone has recently been found which possesses the property of elasticity to such an extent that it may be bent like a thin piece of steel. When a blast is made in the new form of hole the stone is under high tension, and being elastic it will naturally pull apart on such lines of weakness as grooves, especially when they are made as is usually the case in this system, in a direction at right angles with the lines of least resistance.

Our previous illustration of a break by the new system was its simplest and best application. An identical case would be one where a large and loose block of stone was split up into smaller ones by one or more of this form of holes. But those who use this system do not confine it to such cases alone. Horizontal holes are frequently put in and artificial beds made by "lofting." In such cases where the rock has a "rift" parallel with the bed, one hole about half way through is sufficient for a block about 15 feet square, but in "liver" rock the holes must be drilled nearly through the block and the size of the block first reduced.

A more difficult application of the system, and one requiring greater care in its successful use, is where the block of stone is situated, as in the case hereinbefore cited, except that both ends are not free, one of them being solidly fixed in the quarry wall. A simple illustration of a case of this kind is a stone step on a stair-way which leads up and along a wall. Each step has one end fixed to the wall and the other free. Each step is also free on top, on the bottom and on the face, but fixed at the back. We now put one of the new form of holes in the corner at the junction of the step and the wall. The shape of the hole is as shown in Fig. 14.

It is here seen that the grooves are at right angles with each other, and the block of stone is sheared by a break made opposite and parallel with the bench, as in the previous case, and an additional break made at right angles with the bench and at the fixed end of the block. Sometimes a corner break is made by putting in two of the regular V-shaped holes in the lines of the proposed break and without the use of the

corner hole. A useful application of this system is in splitting up large masses of loose stone. For this purpose the V-shaped grooves are

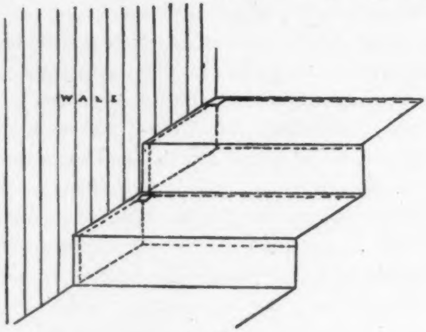


FIG. 14.

sometimes cut in four positions and breaks are made in four directions radiating from the center of the hole as shown in Fig. 15. In this way a block is divided into four rectangular pieces.

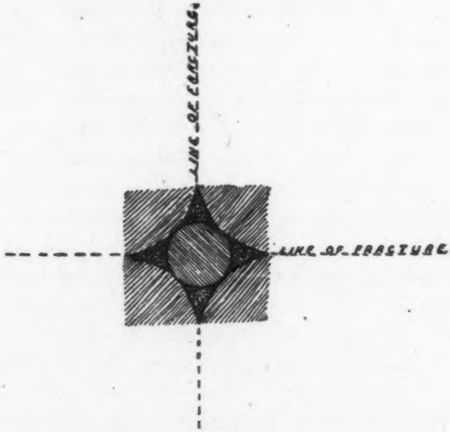


FIG. 15.

Though the new system is especially adapted to the removal of heavy masses of rock, yet it has been applied with success in cases where several light beds overlie each other. In one such instance ten sheets, measuring in all only 6 feet, were broken by a blast, but in cases

of this kind the plug and feather process applies very well, and the new system, when used, must be in the hands of an expert, or the loss will be serious.

Referring again to our stone step, let us imagine a case where this stair-way runs between two walls. We have here each step fixed at each end and free only on the top, the bottom, and one face. Let us assume that there is a back seam, that is, that the step is not fixed at the back. In a quarry, this seam, unless a natural one, should be made by a channeling machine. In order to throw this step out of place it must be cut off at both ends, and for this purpose the V-shaped holes are put in at right angles to the face. It is well, however, to put the first two holes next the back seam in a position where the grooves will converge at the back so as to form a sort of key which serves a useful purpose in removing the block after the blast. In quarries where there are no horizontal beds a channeling machine should be used to free the block on all sides and to a suitable depth, and then the ledge may be "lofted" by holes placed horizontally.

Where "pressure" exists in quarries, the new system has certain limitations. After determining the line of "pressure" it is only practicable to use the system directly on the line of thrust, or at right angles to it. It is much better, however, to release the "pressure" from the ledge by channeling, after which a single end may be detached by a Knox blast. It is well to bear in mind that the holes should invariably be of small diameter. In no case should the diameter of hole be over 1¼ inches in any kind of rock. This being the case the blocks of stone are delivered to the market with but little loss in measurement. Every one knows that the buyer of a block of stone will figure its contents from the minimum measurement of its faces. A hole or groove at the top will shorten the measurement of the faces so that a good deal of stone may be quarried and even shipped to market without benefiting the quarryman, because it does not figure in the measurement. It is a noticeable fact that stone quarried by the new system shows very little evidence of drill marks, for the faces are frequently as true as though cut with a machine.

A matter of no little importance is the safety of the system. The blasting is light and is confined entirely within the hole. No spalls or fragments are thrown from the blast, hence the blasts may be made safely within city limits.

PLATE XCIV.
TRANS. AM. SOC. C. E.
VOL. XXV, No. 510.
SAUNDERS ON QUARRYING.

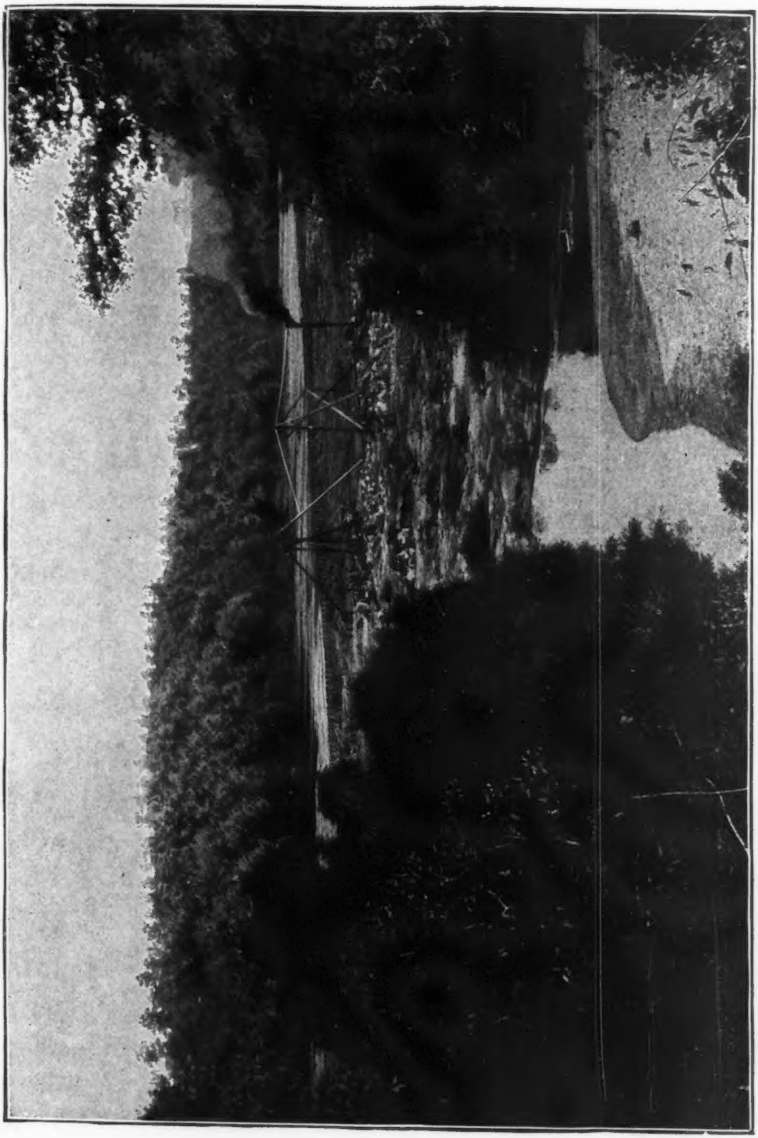
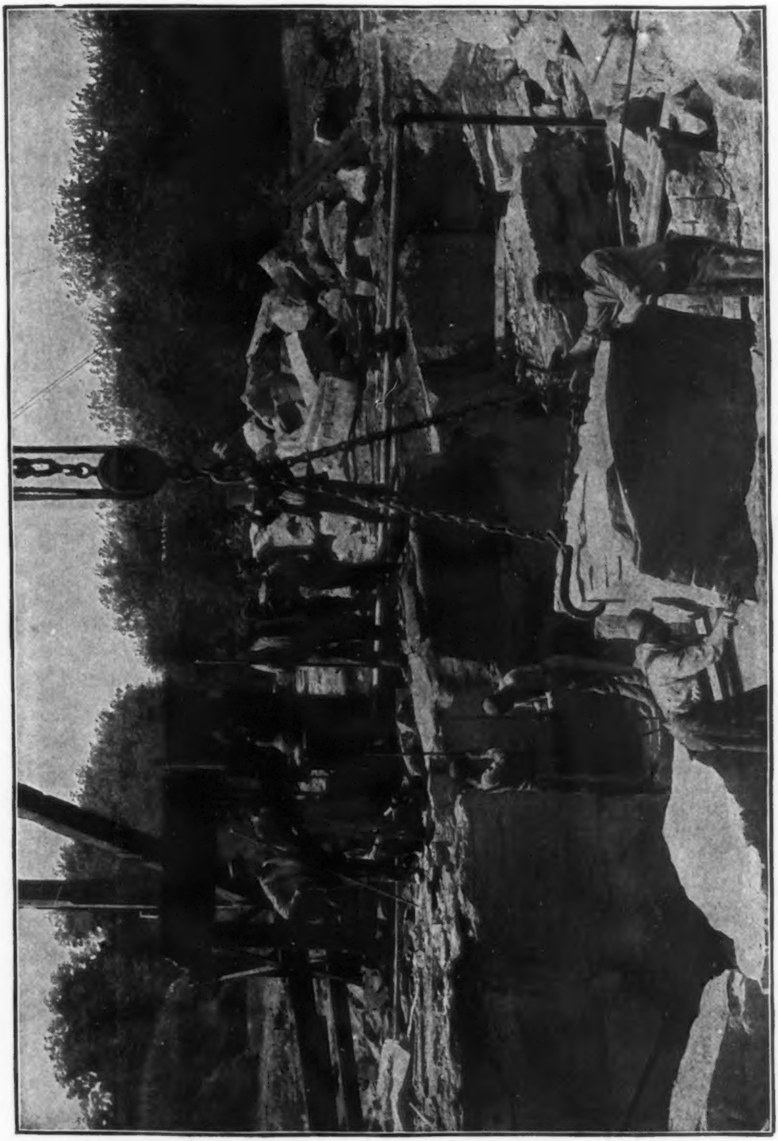


PLATE XCV.
TRANS. AM. SOC. C. E.
VOL. XXV, No. 510.
SAUNDERS ON QUARRYING.



The popular idea that the system is antagonistic to the channeling process is a mistaken one. There are, of course, some quarries which formerly used channeling machines without this system, but which now do a large part of the work by blasting. Instances, however, are rare where the system has replaced the channeler. The two go side by side, and an intelligent use of the new system in most quarries requires a channeling machine. There are those who may tell of stone that has been destroyed by a blast on the new system, but investigation usually shows that either the work was done by an inexperienced operator, or an effort was made to do too much. Most good things are overdone. A quarryman who finds that a simple hole with a small amount of powder will release a large block in good marketable shape, is very apt to carry his enthusiasm so far that he attempts to use the system in places where it is not suited. Blasts are made where there are not sufficient free faces, and a desire to avoid the expense of a channeling machine leads to the destruction of a valuable stone.

A most interesting illustration of the value of this system side by side with the channeler, is shown in the Northern Ohio Sandstone Quarries. A great many channeling machines are in use there working around the new form of holes, and when used together in an intelligent and careful manner the stone is quarried more cheaply than by any other process that has yet been devised.

To a limited extent the system has been used in slate. The difficulty is that most of the slate quarries are in solid ledges where no free faces or beds exist; but it has been used with success in a slate quarry at Cherryville, Pa., since 1888. Among notable blasts made by this system are the following: At the Mica schist quarries, at Conshohocken, Pa., a hole 1½ inches in diameter was drilled in a block which was 27 feet long, 15 feet wide and 6 feet thick. The blast broke the stone across the "rift," only 8 ounces of Dupont black powder being used. At the Portland, Conn., quarries and the Middlesex Quarry Company, a single blast was fired by electricity, fifteen holes being drilled, with 2 pounds of coarse No. C powder in each hole, and a rock was removed 110 feet long, 20 feet wide and 11 feet thick, containing 24 200 cubic feet, or about 2 400 tons, the fracture being perfectly straight. This large mass of stone was moved out about 2 inches without injury to itself or the adjoining rock.

Another blast at Portland removed 3 300 tons a distance of 4 inches. Seventeen holes were drilled, using 2 pounds of powder in each hole, the size of the block being 150 x 20 x 11 feet. In a Lisbon, Ohio, quarry a block of sandstone, 200 feet long, 28 feet wide and 15 feet thick was moved about a half an inch by a blast. This block was also afterward cut up by this system into blocks 6 feet square. A sandstone bowlder 70 feet long, average width 50 feet, average thickness 13 feet, was imbedded in the ground to a depth of about 7 feet. A single hole 8 feet deep was charged with 20 ounces of powder, and the rock was split in a straight line from end to end and entirely to the bottom. A ledge of sandstone open on its face and two ends, 110 x 13 x 8 feet, was moved by a blast about 3 inches without wasting a particle of rock. Eight holes were drilled with a steam drill, three men being employed just one day, and 15 ounces of powder being used in each hole. A sandstone ledge, open on the face and end only, 200 x 28 x 15 feet, containing 84 000 cubic feet of stone, was moved half an inch by twenty-five holes each containing 1 pound of powder.

The reports from the Eleventh Census show that in the year 1889 about $53 000 000 worth of stone was quarried in the United States. Ten years prior to that time, as shown by the Tenth Census, the annual output was $18 000 000. This enormous increase has been made possible by improvements in the quarry which have cheapened the production of stone. Who would not use stone for buildings of all kinds, and who does not prefer it to wood, brick, iron, or any other material? Its great cost alone prevents its almost universal use.

DISCUSSION.

Leffert L. Buck, M. Am. Soc. C. E.—Mr. Saunders' interesting paper on quarrying is of a value disproportioned to its length. The subject is one that has rarely been brought before the Society, and as it is one in which our profession is as deeply interested as in any other, the paper deserves to be thoroughly discussed. Doubtless many of our members, who have not read the report of the tenth census on this subject, will be surprised at the rapid growth of the business in the country in the last few years.

The description of the process called "Lewising," reminded me of the first time I saw it used. While preparing the foundations of the Verrugas Viaduct, in 1872, there was a large boulder of excellent granite to

be broken up, and the foreman (a Scotch stone-cutter and mason) said he would break it by a "lewis-hole-blast." He caused two holes, 1½ inches in diameter and as close together as possible without running one into the other, to be drilled to a depth of about 7 feet. The holes were in the middle of the top and the plane of their axes coincided with that of rupture desired. The partition was broken out. Ordinary black powder was used and the break was almost an exact plane. It is not difficult to understand why the blast should act in this manner. There is greater surface for the pressure to act upon in one direction than in that at right angles to it, and of course the rupture will be parallel to the surface of greatest pressure, where the blast is about in the center of a stone of homogeneous structure. It is true that the effect of a three-cornered hole, such as an ordinary hand drill is apt to make, is to cause a blast to break the stone into three parts generally. This is also apt to take place where the hole is round, unless the greater part of the rock is on one side of the hole. I witnessed a fair proof of this in Pittsburgh in 1878. A "salamander" had been allowed to form in the bottom of a furnace stack and was broken by blasting. Roughly speaking, its shape resembled an army canteen and was 6 or 7 feet in diameter by about 2 feet thick in the center. A cylindrical hole was drilled in the center to a depth of about 17 inches with a ratchet drill. It was charged with dynamite, and broke into three nearly equal parts. There were slight incipient checks between the breaks which I was never able to account for satisfactorily. But that the piece should break in three places, is of easier solution. Of course we know it would break in the way that presented least resistance. As far as resistance to rupture alone is concerned, it would favor the piece breaking in two pieces, on or about some diameter, for the reason that each of the two fractures presents no greater resistance than each of the three, while the areas of the bursting pressures are respectively as 1 to about .866; moreover the direction of the bursting pressure is more favorable to the two fractures than to the three. But when the breaking is effected by a high explosive, the resistance of inertia comes in, and as this is but two-thirds as great in the three-fracture case as it is in the two, it becomes sufficient to outweigh all the other advantages which would favor the two fractures, and causes the three. Hence the higher the explosive energy the greater the number of fractures. In all cases the breaking is a tearing process from the hole outward. If this reasoning is true, it accounts for the advantage of the air cushion in quarry blasting. It might be questioned whether a still slower burning powder would not prove of additional advantage.

The census report referred to gives some statistics of cost of quarrying which are valuable.

M. W. Mansfield, M. Am. Soc. C. E.—In 1886, we advocated a trial of the Knox system of quarrying in the oolitic limestone fields of Indiana.

It did not meet with favor. I am of the opinion that this system will not work in oolitic stone for the reason that this stone is without seams and homogeneous and is highly elastic. All oolitic stone is now quarried with channelers and the block of stone is raised from the bed by driving small wedges in holes made with steam drills. These holes are usually 6 to 8 inches apart, 6 inches deep, and the time consumed in drilling each hole is about eighteen seconds. Skilled labor is not required in this work. I do not think the Knox system can be used advantageously and with economy in connection with channelers, but consider it an improved system for blasting which can be used only in quarries where blasting is permissible.

R. C. McCalla, Jr., Assoc. M. Am. Soc. C. E.—The Knox system of blasting has been in use since September 1st, 1891, on the Black Warrior River improvement at Tuscaloosa, Ala., in a quarry which probably could not be operated at a reasonable cost without the aid of this method. The quarry, a bird's-eye view of which is shown in Plate XCIV, is located on falls, in the bed of the Black Warrior River, adjacent to the sites of four locks which are now being built by the United States Government. This is the only quarry located in the bed of a stream of which I have ever heard, and was opened partly as an experiment and partly from necessity. The old quarry, located three-quarters of a mile above the present one, on the bank of the river above high water, had given out. The stone had become seamy and the stripping very heavy. There was no other available quarry site near the work and above high water, so that stone had either to be brought by rail from a distance or gotten from the river bed.

Lock 3, not yet begun, is located just above the present quarry, and considerable channel work is necessary for the lower approach to this lock, and as a coffer-dam had to be built and considerable work done at this point, in any case, it was decided to extend the dam further down stream, and by thus laying the bed of the river dry, make it available as a quarry.

The coffer-dam is 5 feet high, and, starting from the west bank just above the site of Lock 3, runs 200 feet across the current of the stream, which is 700 feet wide at this point; and thence 1 000 feet down and parallel to the current, to the head of an island which acts as an extension of the dam. The fall from the head of the dam to the pool below is about 7 feet at low water, and the area of river bed thus exposed is about 7½ acres.

The stone is a very fine quality of light blue sandstone, lying in ledges from 2 to 8 feet thick, which dip down stream at an angle of 7 or 8 degrees. In quarrying, a ledge is taken at its outcrop and followed down stream until it gets too deep to be worked to advantage. Then the quarry is stepped up to the next ledge above, which is followed in like manner.

There is considerable seepage water, which is controlled by a 3-inch

pulsometer pump located out of the way in one corner of the excavation and moved forward from time to time as the ledge gets deeper. Still, it is impossible to get rid of all the water immediately at the breast of the quarry, as the depth is constantly changing and varies at different points along the line of breast. As a matter of fact the stone is constantly quarried in from 1 to 4 feet of water, and this accounts for the great importance of the Knox system of blasting in this particular quarry. There is no chance to spring a hole, and whatever is done must be done by a single charging of the hole, the water entering immediately after the blast.

In a dry quarry it is practicable to detach a large block of stone, generally in some irregular shape, by repeated blasts in the same hole, beginning with a light charge and gradually increasing the charge until the block is separated from the mass, when enough powder may be placed in the crevice to throw the block entirely out of place without injury. It can then be cut up with ball drills, plugs and feathers. Were enough powder put in the old style of hole to detach the block at the first blast it would be almost certain to shatter the stone; and for this reason I do not think this quarry could be worked except by the Knox system without very great waste. Plate XCV gives an excellent view of a portion of the quarry breast.

At present, owing to the lack of machinery, all drilling in the Warrior River quarry is done by hand; but next season it is intended to introduce steam drills, which will reduce the cost of stone considerably. All drills used have the cross-bit, with two cutting edges at right angles to each other. A hole drilled by hand with a flat bit will invariably be somewhat triangular in cross-section, and a blast in such a hole will usually break the stone in three directions radiating from the corners of the triangle, resulting in more or less waste of stone; but there is not the slightest difficulty in drilling a round hole by hand if the cross-bit is used.

During the month of October, 1891, 200 cubic yards of backing and 600 cubic yards of dimension stone were taken from this quarry at a total cost of $1 597.73. Estimating the cost of backing at $1 per cubic yard, the dimension stone cost $2.33 per cubic yard. The quarrymen are negroes and are paid $1 for eight hours' work.

William L. Saunders, M. Am. Soc. C. E.—Mr. Mansfield, of Indiana, is quite correct in his statement that the Knox system of quarrying will not work to advantage in oolitic stone. He is, I think, mistaken in giving as one of the reasons for this that oolitic stone is without seams, homogeneous and highly elastic. It is not an uncommon thing to use the system in quarries where there are no seams. The channeler is used to make an artificial seam, which, in connection with the Knox hole, enables the quarryman to remove large masses of stone in marketable shape. The same objection to the Knox system of blasting in oolitic stone applies to any system of blasting whatever in soft stone. It is a well known

fact that powder is not generally used with success in removing banks of clay, because the material does not resist the force of the explosive, but it simply gives way around the hole, throwing out a mass of material and leaving a pocket in the shape of an inverted cone. A blast in a drill hole acts like a blow from a sledge hammer, and the higher the grade of the explosive the greater is the blow. We may strike a piece of pasteboard with a hammer and produce only a local effect, while the same force of blow, or even less, when applied to a piece of glass will shatter it to pieces.

Oolitic stone is quite soft and the easiest stone to channel that a machine has to encounter. It is not an uncommon thing for a channeling machine to cut 200 or 300 square feet of channel in oolitic per day, while the capacity of the same machine in marble is less than half as much.

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THE BRAZOS RIVER HARBOR IMPROVEMENT.

By GEORGE Y. WISNER, M. Am. Soc. C. E.

WITH DISCUSSION.

The Brazos River and the harbor at its mouth have played a very important part in the development of the State of Texas, since the earliest settlements on the coast by American colonies; and the success which has recently been attained there in making it the deepest harbor on the coast, is likely to give it much greater prominence in the future. In 1824 General Austin made the harbor a port of entry for the colony. The commerce soon became of considerable importance and continued to increase until 1858, at which time a large amount of the river traffic was diverted to Houston over the railroad built west from that city.

During the days of the Republic, about 1845, a number of prominent citizens of Texas formed a company for the purpose of making a harbor and building a city at the mouth of the river. Nothing, however, was ever accomplished.

In 1857 the Legislature of the State, realizing the necessity of a first class commercial port on the coast, appropriated $60 000 to deepen the channel and improve the river from the mouth to the town of Wash-

ington, a distance of 250 miles. The improvement accomplished was only temporary, and a few years later the erection of railroad bridges across the river at several places, completely blocked navigation above tidewater. About the same date that the above improvement was undertaken, a company was formed and a charter obtained from the State Legislature for the purpose of deepening the entrance of the harbor and making it the eastern terminus of the Pacific route then being surveyed. Owing to financial trouble no work was ever done under this charter.

By an act of the Legislature of Texas, passed October, 1866, the Brazos Internal Improvement and Navigation Company was incorporated in the interest of the Houston and Texas Central Railway, for the purpose of improving the Brazos River for navigation by deepening the channel across the bar at the mouth, building a breakwater and dredging wherever necessary. The capital stock of the company was $1 000 000 and the charter was to continue for fifty years. The company had the power under its charter to condemn lands lying contiguous to their work, and to charge tonnage at a rate not to exceed twenty cents per ton of freight. Nothing further was done in the matter until March, 1872, when the owners of the Houston and Great Northern Railroad, then being constructed between Houston and Palestine, decided to join the Houston and Texas Central Railway people in the work of making a deep water harbor at the mouth of the Brazos River, and accordingly certain of the old directors resigned and a joint directory was formed from the stockholders of the two companies. The Houston and Great Northern Railroad obtained from the State Legislature a subsidy of $10 000 per mile, but this the following Legislature, in 1874, refused to recognize, on the ground that it had been procured by bribery. The failure to obtain this subsidy together with the financial panic of 1873 so crippled the company that no improvement work was attempted at the harbor. At the request of this company the Secretary of War granted a leave of absence for Captain C. W. Howell, of the U. S. Engineer Corps, to examine the bar at the mouth of the river, and submit plans and an estimate of cost for making a ship channel 20 feet deep from the river to deep water in the gulf.

Captain Howell was very sanguine of success, and in his report says:

"For a short time after the improvement of the bar, the harbor facilities afforded by the river will be sufficient, but it is, however, certain that the success anticipated for the work of improvement, giving the deepest entrance to the Brazos of any harbor on the Texas coast, must

attract a commerce that will rapidly outgrow the limited harbor room. You should, therefore, look forward to a system of docks communicating with the river by suitable locks, so as to have the whole river channel free for the passage of vessels up and down."

The plan submitted consisted of two jetties of closely-driven piles, starting from the headlands, at the mouth of the river, and converging so as to give an opening of 400 feet between the outer ends of the jetties on the crest of the bar, and estimated to cost $286 484. It was, however, fortunate for the promoters of the enterprise that the work was never undertaken, for, under the plans submitted, it would have been impossible to have carried it to completion. The teredo is so destructive on the gulf coast that piles driven in the open sea will not stand more than one season if unprotected, and the volume of discharge is such that the mean velocity of the current between jetties 400 feet apart would have exceeded 7 feet per second at times of floods in the river. Such a structure as proposed, subject to the action of the sea and river currents, would probably have been swept from its unstable foundation in a single night.

In 1874 Congress authorized a survey of the river from Waco to its mouth and including the bar. This survey was made during the years 1874 and 1875 by Captain R. B. Talfor, under the direction of Captain C. W. Howell of the corps of engineers. Captain Howell in his report on this survey repeats his recommendations made to the Houston and Great Northern Railroad Company in 1872, and submits the same plan for improvement then given. No further action was taken in the matter by the Government until 1880, when Congress made an appropriation of $40 000 to commence the work of construction in accordance with the plans of the U. S. Engineer Corps. The project was then referred to a board of engineer officers for revision, who discarded the old plans and recommended building parallel jetties of brush and stone, 700 feet apart, and estimated to cost, when completed, $522 890.44. The conclusion reached in the report was that possibly a channel 10 feet deep could be made and maintained across the bar.

In December, 1880, a contract was awarded for carrying the board's plan into execution, under which brush was to be furnished in place in the jetties for $6.75, stone ballast for $8, and small concrete blocks for $7 per cubic yard. Most of the work done under this contract consisted in revetting the east bank of the river, inside of the shore line, with brush mattresses ballasted with gulf sand. In 1881 Congress appro-

priated $40 000 more to continue the work, and another contract was awarded, under which brush mattress work was to be placed in jetties at $4.35, and gravel and concrete ballast at $7 per cubic yard. Under this contract the work on the east jetty was continued, but from all accounts very little of it remained long in place. In 1882, 1884 and 1886 $50 000, $10 000, and $18 750, respectively, were appropriated for continuing the work, all of which was expended in extending the east jetty, except a small amount of work in 1883 on the west jetty, which consisted of brush mattresses ballasted with plastic concrete made of natural cement in bags. The deterioration of the jetties was so rapid that in 1887 the engineer officer in charge reported that "the works heretofore constructed have disappeared, and have had no effect upon the bar." In other words twelve years of labor and an expenditure of $158 750 had produced no permanent result.

A survey and examination of the harbor entrance was made in 1887 by one of the most competent civil engineers on the gulf coast, who reported to the officer in charge that with a proper construction of jetties, a channel 16 feet deep could easily be made and maintained across the bar. Disregarding this the engineer officer reported after a long discussion that,

> "With jetties 700 feet apart, the depth scoured by the greatest freshets would be something less than that found within the mouth of the river. It could hardly exceed 10 feet. How long it would take this depth to decrease to less than 6 feet, and whether the frequency of freshets is such that the necessary time would never be found, cannot be stated. All that can be said is that the attempt to maintain 6 feet is a very doubtful experiment. A study of the subject in the light of additional information shows the degree of improvement needed is impracticable, and that the attempt to make any improvement at all is a costly and doubtful experiment."

The officer seems to have relied entirely upon data in previous official reports which were very meagre and misleading. These state among other things that the river is a non-sediment bearing stream, because there is no delta at its mouth ; and that at low river it is like a series of pools separated by sand bars, which are often dry. Nothing could be further than this from the real facts of the case. The lower 50 miles of the river is a tidal stream, which for a low stage of water has a channel 15 to 50 feet deep for over 30 miles from its mouth. The total length of the main river is about 1 000 miles, and with its tributaries it drains an area of 36 000 square miles. The annual rain-fall along the coast is from 50 to 60 inches, in the interior 30 to 40 inches,

and for the whole drainage basin averages not far from 42 inches per annum. It is not probable that more than one-seventh of the rain-fall on the drainage basin finds its way into the gulf through the river, making the annual discharge about 501 811 200 000 cubic feet, or an average of 15 900 cubic feet per second. If this flow was a constant one, the problem of improvement would be a very simple one; but such is far from being the case. The period of low water in the river is of from one to three months' duration each year, during which time the discharge is only 1 000 to 3 000 cubic feet per second, while at the flood stage the discharge often exceeds 60 000 cubic feet per second. The amount of sediment carried in suspension by the river, varies from none at low water, to 4 ounces per cubic foot of discharge during freshets. The sediment is mostly fine alluvion, with a very small amount of sand.

The banks of the river are comparatively stable except on the lower 26 miles of its course, and nearly all the sand carried to the gulf by the freshets comes from the caving of banks on this lower reach. These banks are of clay for 10 to 20 feet in depth, resting on strata of fine sand. During high river these strata of sand become saturated, and when the pressure is relieved at low water the sand sloughs out and with the next rise is swept on towards the gulf. The banks, however, are of such a character that this trouble could be easily remedied, which would limit the character of sediment to that of such fine material that it would be swept far out to sea before finding a permanent resting place on the gulf bottom. The fineness of the sediment deposited on the bar and the strong littoral currents in the gulf, are the real reason that there has never been any permanent advance of the bar seaward, nor any delta formation at the mouth of the river.

The report of the Chief of Engineers for 1875 states that, "The floods and the 'northers' completely sweep away that portion of the bar formed during low water in the river, and drive the material so swept off into the deep water of the gulf." This is really the reverse of what does occur. The bar in its natural condition is formed by sediment deposited by the river during floods and by sand drift moving westward along the coast by the current and wave action. The past history of the harbor shows that the depths on the bar have varied but little except in cases when violent storms have closed the main channel, leaving depths of only 3 to 4 feet, which in some instances have lasted several

weeks before the river could break through the barrier in some new place. The extension of the bar seaward during freshets was not due to a pushing out of old bar material, but simply to an accumulation on the outer slope from the excessive load of sediment which the slackened currents were unable to carry further.

The low water season of fall and winter is also that of heavy easterly winds, during which the littoral currents westward are from one to three miles per hour. Under the action of the waves and of these currents the outer face of the bar has been annually worn away and the material distributed over the gulf bottom to the westward. This result is very plainly shown on the charts of the coast by the bending seaward of the 10 and 20 fathom curves immediately to the westward of the entrance of the harbor. In 1858, the 18-foot contour outside of the bar was 1 200 feet further seaward than in 1881, whereas after the big rise in March, 1889, when the present work was commenced, the position of the bar was approximately the same as in 1858.

Borings on the bar show that to a depth of 21 feet below mean gulf level it is composed of fine sand and shell. Below this sand and shell there is a stratum of clay about 2 feet thick overlying another of sand of 8 feet thickness, on a heavy clay bed, having about the same slope seaward as the gulf bottom. The bottom beyond the bar has approximately a slope of one in five hundred, and is mostly of clay and soft mud deposit from the river. During storms this soft material is disturbed to such an extent that the gulf becomes the color of the river at flood stage and causes changes of depth such that consecutive surveys often show differences of 1 to 2 feet over large areas.

The movement of the sand drift to the westward does not extend much outside of the 18-foot contour, and previous to the construction of the jetties had a tendency to narrow the entrance of the harbor during the low water season, by building up a sand point at the east side. The sand thus accumulated was swept out with each rise in the river, thus causing the cross-section of the entrance to be somewhat variable. Observations made when the entrance was thus choked by this drift action, led a recent board of engineers to the conclusion that the cross-section of the harbor entrance was so small that it was not worthy of consideration. Since the construction of jetties into the gulf this sand drift accumulates in a high concave sand beach to the east of the works, while

the material scoured from the river forms a convex bar to the south of the entrance.

The force of the gulf currents depends greatly on the direction and velocity of the wind. With strong easterly winds the currents are from 1 to 3 miles per hour westerly, while winds from the south and west produce very light surface currents in the opposite direction. The former, however, are the only ones of sufficient force to produce any material change in the conformation of the bar and of the gulf bottom beyond. The magnitude of these forces becomes very evident when we consider that eight to twelve million tons of sediment are annually deposited in the gulf from the river, and that no permanent change of the deep water contours occurs in front of the harbor.

The banks of the river for a distance of 50 miles up stream have approximately the same slope as that of the river at flood stage. The width of the river at 25 miles from the gulf is about one-half that at the mouth, and as the depth at flood stage is twice that at the entrance, the cross-section for the maximum discharge is approximately the same. This fact establishes the section of channel that can be made and maintained between properly constructed jetties at the mouth.

The slope of the river surface is not affected by the distance that the jetties are extended seaward, the only effect being an additional rise in the river for any given stage, equal in amount to that due to the slope in the jetty channel. With tidal harbors, however, the slope of the surface in the jetty channel is inversely as the length of the pass, and consequently on the gulf where the tides are very small (about 1 foot) and the distance to deep water very great, the plan of improving harbor entrances by confining tidal currents between jetties is a somewhat doubtful experiment.

The project now being carried out to make a deep water harbor at the mouth of the Brazos without Government aid was originated by Mr. W. M. D. Lee of Kansas, to whose enterprise and energy the success thus far attained is to a large extent due. Mr. Lee first visited the harbor in 1887, soon after the adverse report was made by the Government engineer, and was so thoroughly satisfied of the importance and feasibility of making a first-class harbor there, that during the following winter he secured the control of a large amount of land contiguous to the river near its mouth; and in March, 1888, organized the Brazos River Channel and Dock Company, and obtained a charter from the

Texas Legislature for the purpose of constructing, owning and operating a deep water channel from the waters of the Gulf of Mexico to the mainland at the mouth of the Brazos River. Mr. E. L. Corthell, M. Am. Soc. C. E., was employed to report on the feasibility of the project and to submit plans and estimates for doing the work. At Mr. Corthell's request, the writer, in March, 1888, made an examination of the harbor and vicinity, from which, with data obtained from the office of the Government engineer, his report was compiled and plans and estimates made. The only data of value obtained from the U. S. Engineers were a few comparative charts of the harbor entrance, and the elevations of the bench marks for a line of levels extending up the river 16 miles from the mouth. A very fortunate time, however, was chosen for making the examination, and the data then obtained have been subject to but slight corrections by the large number of careful observations since made.

The river at the time of examination was at about one-half flood stage; the plane of high water surface being well defined by marks on the timber along the banks. From a careful determination of mean velocity, discharge and slope, the co-efficient of roughness for "Kutter's formula" was determined for the section of the river used, from which, with the slope for high water, the maximum discharge was computed. Subsequent observations have since shown this determination to be in error less than one-half of 1 per cent.

The plan recommended for the improvement of the harbor was the construction of parallel jetties 600 feet apart, and extending into the gulf to a depth of 18 feet outside of the bar. The works were to be of brush mattress work ballasted with stone, and were expected to make and maintain a channel with a central depth of at least 20 feet. Mr. Corthell proposed in this report to construct the jetties from a trestle to be built seaward as the work progressed. The mats were to be built on tilting ways placed on a platform car, to run on a double track railroad laid on two parallel pile bridges, the piles being driven by an overhanging driver.* This plan, however, was not practical, as the mattress car would occupy the entire width of the trestle and consequently could not pass the driver to dump the mats at the end of the trestle as required. The delays which would arise from trying to carry on pile

* See *Engineering News*, June 22d, 1889.

driving and jetty construction in so limited a space would have made the progress very slow and the work very costly.

During the summer of 1888, the plans and method of construction were therefore changed by the writer. The system adopted consists in suspending the mat during construction, directly beneath the caps and stringers of the trestle, and when completed allowing it to slide down the piles of the trestle to its place on the gulf bottom, by simply loosening the suspending ropes attached to the caps and stringers. This method had the advantage over those previously used, that all work was built in place. Material could be transported over the trestle at all times, and the work carried on over any extent of jetty desired. On all work inside of bars, where not subject to heavy wave action, clay, shell, or gravel may be used for ballast and placed directly in the mats before binding, thus effecting a great saving of stone ballast, and at the same time making the jetties more impervious and safe from the destructive action of the teredo.

In August, 1888, a bill was passed by Congress authorizing the company to construct, own and operate such permanent and sufficient jetties and such auxiliary works as may be necessary to create and permanently maintain a navigable channel between the mouth of the Brazos River and the Gulf of Mexico. In December following, a contract was let to construct the jetties and create a 20-foot channel between the gulf and mouth of the river—the contractor to receive payments only as certain results were obtained. The work was to be done in accordance with the plans and specifications of the Chief Engineer of the company, but the only stipulation made as to the material to be used and the character of work, was that the jetties and auxiliary works were generally to be similar to those at the mouth of the South Pass of the Mississippi River. To the weakness and anomalous features of this contract may be ascribed the trouble that has harassed the company for two years and which on two occasions has nearly wrecked the enterprise. The fact that the contractor was to be responsible for results, should have carried with it the right to originate, design and construct the works, so long as the material used and work done were of a permanent character. On an enterprise of this kind, where unfinished work was liable to damage at all times, and the results expected were dependent on unknown quantities; with the design and direction of the work resting with one party, and the responsibility for results with another, a conflict was inevitable.

Mr. E. L. Corthell was appointed Chief Engineer for the Company, and in March, 1889, the writer took charge of the work as Resident Engineer. The change of plans made by the resident engineer by which the mats were constructed in place, and clay and shell used for ballast for the half mile of jetties inside of the bar, made a saving to the contractor of upwards of $25 000. The mats were constructed from a trestle of four rows of piles (spaced laterally to correspond with the width of the jetty) in bents of 16 feet each, with stringers of 10 x 10 inch timbers resting on caps of 8 x 10 inch timbers drift-bolted to the top of the piles. The timbers used for binding the mats were of 2½ x 6 inches, made continuous by splicing, and during construction were supported above wave effect by ropes made fast to the caps and stringers of the trestle. Sufficient brush was piled crosswise of these strips for half the thickness of the proposed mat, and it was then loaded with enough clay and shell to sink the entire mat. The brush for the completion of the mat was then placed at right angles to the former course and the whole firmly compressed between the mattress strips, with binders of wire or iron rods. The latter are preferable, but in this work a large amount of wire rope was used for the reason that the contractor had the material on hand and it was worthless for any other purpose.

Mattress work made in this manner is flexible, and when not subject to strain from wave action may be made continuous for any length, by simply lowering the mat by degrees as completed, so that it forms an incline from its final position in the jetty, to the unfinished portion on the suspending timbers. Where the mats were liable to be disturbed during heavy storms they were made in lengths of 100 to 300 feet, and allowed to sink quickly by loosening all of the suspending lines simultaneously.* Outside of the bar, where the force of the waves was sufficient to wash the clay and shell ballast from the mats, only stone was used for ballast, and it was generally placed on top of each mat after being completed.

No trouble was experienced on the half mile of work lying inside of the crest of the bar, but beyond this the force of the waves was such that a much more substantial structure was needed, and the contractor was surprised at the ease with which mats ballasted with 600 pounds of sandstone to each cord of brush were removed from the jetties by the

* See *Engineering News*, May 18th, 1889, for plates descriptive.

waves and the material distributed for miles along the shore. The clause of the contract requiring the works to be similar to those at South Pass, was interpreted to mean that they should contain one-third of a cubic yard of stone for each cord of brush, and while admitting the necessity of more substantial work, the contractor claimed that he was not legally required to do differently, and that all losses arising from weakness of the structure must be made good by the company. Under this state of affairs but little was accomplished, and after completing the jetties to the water surface for a distance of 3 000 feet from shore, all work was discontinued. The trestle was, however, unfortunately completed to a distance of 5 000 feet from shore on the west jetty, and 4 000 feet on the east jetty, which latter having no mattress to protect the bottom from scour, the waves cut a trench along the line of open piling, of such a depth that the amount of mattress work finally required was nearly double that which would have been necessary, if the work of jetty construction had been carried on simultaneously with that of the trestle.

In the latter part of June a heavy rise occurred in the river, and although at that time only 800 feet of jetties were in place, a channel 20 feet deep was scoured through the completed portion ; but the end of the works terminated so far inside of the crest of the bar, that the only result was to push the bar several hundred feet further seaward than was done by the rise of the previous winter.

During the summer an attempt was made to float the bonds of the company on the English market, and Sir John Coode, President of the Institute of Civil Engineers, was requested to examine the project and report on the probable result of the enterprise. Mr. J. C. Coode made a personal examination of the harbor and the plans for its improvement in December, and from the data thus obtained Sir John Coode made an elaborate report, in which he says :

"I consider the system on which the Brazos jetties have been laid down by the engineers of the company to be perfectly sound. The points on which I differ from these gentlemen are : First, the probable extent to which it will be necessary to project the jetties into the gulf ; and second, the amount of contraction requisite to maintain the standard depth throughout the year. The navigable depth, which is in my opinion the maximum likely to be maintained throughout the year by means of the works as proposed, has already been stated as 20 feet at mean low water, and a width of channel at that depth of 100 feet."

Mr. Coode was of the opinion that the jetties should be built out to the depth of 21 feet in the gulf in order to secure a permanent naviga-

ble channel of 20 feet, and that the entrance should be contracted to a width of 350 feet. Subsequent events have shown that a channel of such dimensions would not carry the volume of water discharged at the flood stage of the river without endangering the stability of the jetties, and that although the 20-foot channel has not yet been secured it is quite evident that the jetties will not have to be carried out to any such distance as he suggests. The report as a whole was very flattering, but unfortunately before it was made public the stringency of the English money market became such that no use could be made of it for the purpose intended.

At the end of the year the records of the engineer's office showed that there had been expended on the jetties, wharves and wing-dams $218 752, in addition to which the plant and buildings belonging to the contractor were claimed to have cost $70 198, or a total of $288 950, not including the personal expenses of the contractor. Of this amount, however, at least $50 000 may safely be charged to the account of lost material and worthless plant. The contractor's books showed a total expenditure of $327 000, or a difference of $38 050 in the two expense accounts for the same work. A compromise was finally made in January, 1890, by which the entire works and plant were transferred to the company, and a few weeks later Mr. Corthell resigned his position of Chief Engineer, after which the engineering and the entire management of the harbor and jetty construction were placed in the charge of the writer. An entire reorganization had to be effected, new plant and material obtained, and as the season of heavy easterly storms was at hand, but little was done for a month and a half except to make the work already in place more secure by loading the exposed portion with large rock. The trestle that had been built beyond the ends of the completed mattress work was such a wreck as to be of no use for future construction, and it had caused the waves to excavate a trench 16 feet to 18 feet deep for its entire length, where the normal depths would have averaged less than 10 feet previous to the construction. The outer face of the bar had cut away during the winter until it was approximately the same as when the work was commenced the previous year, with a channel depth on the crest of 8 feet at mean gulf level. After the completion of repairs to the old work, every effort was made to push the construction on the east jetty as rapidly as possible, so as to stop further scour along the line of the old trestle, concentrate the river current on the bar, and

form a breakwater for the protection of work on the west jetty from easterly storms.

In the latter part of April one of the heaviest rises in the river ever known began and continued for a period of six weeks, and during twenty days of this time the discharge exceeded 60 000 cubic feet per second and carried in suspension 400 000 to 600 000 tons of sediment per day. In addition to this vast amount of material, over a million cubic yards of clay and sand were scoured from the bed of the channel at the mouth of the river and through the jetties, but owing to its heavy character this was not carried by the currents much beyond the bar. The result of this rise was a channel 25 to 33 feet deep from the inside harbor to the outer end of the completed mattress work, 20 feet for 1 500 feet beyond, and 13 feet deep across the bar to the gulf. The face of the bar was built seaward 500 feet by the deposit of material on the line of the east jetty, and 1 300 feet on the line of the west jetty, making the direction of its front about forty-five degrees with that of the jetty channel. Beyond the bar, shoaling of one to two feet extended seaward nearly a half mile. To the east of the jetties no shoaling whatever occurred, showing beyond question that the gulf currents during the entire time of the freshet were to the westward. This is an important point, not only in regard to the problem now being solved at the Brazos, but also to that of every harbor on the coast.

During the months of May and June the east jetty was completed to a distance of 5 000 feet from shore, and work was stopped on the outer edge of the bar in a depth of 14 feet, and 500 feet inside of the 18-foot contour. In July and August the east jetty was ballasted with rock of 1 000 pounds to 3 000 pounds weight, and the mattress work of the west jetty completed to a distance of 4 000 feet from shore, and with the bottom course of mats to 5 400 feet, opposite the outer end of the east jetty. The first of September, the failure of the Potter Lovel Company of Boston, through which the Brazos Company transacted its financial business, completely demoralized everything and the works passed into the hands of receivers. Funds could not be obtained to complete the mattress work and build the concrete coping walls which the engineer had hoped to have in place in time to protect the jetties during the season of winter gales, and the entire force was disbanded except a small repair crew which it was absolutely necessary to retain to strengthen weak places in the work as they became apparent after storms. Before

financial relief was obtained, it finally became necessary for the Chief Engineer and his associates to put up their private funds for pay of labor to save the works from wreck, and even then the loss from storms was heavy. The jetties, as far as completed, were maintained above the plane of average tide until into March, 1891, when a series of heavy gales washed a large amount of the stone from the crown of the east jetty, leaving depths of 1 to 2 feet of water over the outer 2 000 feet of the work.

No rise of the river of any importance occurred during the fall and winter, and all changes of the bar were due to low river currents and littoral currents in the gulf. Previous to the extension of the west jetty in July and August, the channel across the bar under the action of southerly winds, which are almost constant at that season, shoaled to 11 feet, but no changes worthy of note occurred on the outer slope of the bar. When mattress construction was carried out to the crest, this shoaling was stopped and the channel under the natural forces of the river again deepened to 13 feet. With the commencement of easterly winds in September, rapid erosion became apparent on the outer face of the bar, and continued throughout the winter to such an extent that the 20-foot curve was only 100 feet outside the jetty entrance, and the depths beyond were from 1 to 2 feet greater than at any time since the commencement of the work in 1889. The bar to the south and west of the west jetty was not entirely cut away, and probably never will be. The charts of the South Pass entrance of the Mississippi River show a similar bar extending 1 500 feet beyond the jetties, and at the Sulina mouth of the Danube similar results are apparent, though not to so great a degree.

In its original state the river was about 700 feet wide at its mouth and 14 feet deep, shoaling toward the gulf to 6 feet on the bar, and deepening up stream to a depth of 45 feet at the bend of the river, one-half mile from the mouth. The river channel above the jetties has been corrected to an average width of 475 feet by wing-dams, so located that the new channel forms a tangent to the curve in the river above, thus avoiding any liability of a cross-over bar being formed. This straight reach of river extending to the outer end of the jetties, a distance of 1½ miles, is such that the momentum of flood discharge of the river (after removal of the bar) will probably produce scour for at least 1 500 feet beyond the end of the works.

To prevent undermining by the formation of a deep channel near the jetty walls, spur dikes 40 feet long were constructed at intervals of 400 feet along each side of the jetty channel. The deposits produced by these spurs have built solid walls of mud along each jetty, making them impervious and safe from any danger of undermining or destruction by the teredo.

The inner edge of the bar in its original state was about 3 000 feet from shore on the channel line and united with the shore a half mile west of the entrance. Soon after the completion of the west jetty to the bar, the portion to the westward of the works was built up by deposits and wave action and became a permanent shore line. The pool formed by this new shore line and jetty soon silted up so as to be bare at low tide, thus making the inner 3 000 feet of the west jetty practically a river bank.

No river deposit has accumulated to the east of the works, but the movement of sand drift along the shore has formed a high concave beach in the angle of the east jetty, and extending 1 200 feet seaward. Sand is banked against the outside of the jetty for 3 000 feet from shore, but slopes down to the original depth at a short distance from the works.

The method of mattress construction for 1890 was similar to that of the previous year, except that half-inch iron rods were used for binders instead of wire. Compression was given to the mats after construction, by passing the ends of the rods through holes in the upper binding strip until a hold could be obtained with a shackle bar, with which pressure of about one ton was applied to the mat by lifting the rod through the strip, and afterward fastening it firmly in position with an iron wedge driven into a washer resting on top of the mattress strip. The piles for the trestle were driven with an ordinary overhanging driver of sufficient reach to drive bents 16 feet in advance of the completed structure. With average weather, three bents (48 lineal feet of trestle) were completed per day, and including lost time cost eighty cents per lineal foot for construction. From an average for the season, it was found that the labor required in the construction of mats was one and one-third hours for each cord of brush used, and one hour for each ton of rock distributed in place in the jetties. The brush and the rock were delivered alongside the works on barges. Two cubic yards of brush as

measured on the barges, when compressed in the mats and consolidated with heavy stone ballast, made one cubic yard of completed jetty work.

The following table gives the cost of one cubic yard of completed jetty as determined from an average for the season on the outer 2 000 feet of the works where exposed to heavy sea action:

Trestle	$0 20
Scaffold and mattress frame	0 05
Brush	0 70
Labor on mat	0 10
Rock ballast	1 40
Labor placing ballast	0 08
Engineering and superintendence	0 07
Cost of 1 cubic yard in place	$2 60

The brush cost $1.50 per cord delivered on board of barges, and the rock ballast $3.50 per ton on barges alongside of the works.

In April, 1891, a reorganization was effected by which the contractor, who on retiring from the work the previous year, stated that the enterprise was practically a failure, again took charge of the management and the writer resigned his position of Chief Engineer. The work is so far advanced and the results so satisfactory that no change of plans is contemplated. Mr. A. E. Kastl, M. Am. Soc. C. E., who was assistant on the work during the first year of construction, has been appointed engineer, and will carry out the original plans as far as the means placed at his disposal will allow.

No rise in the river of sufficient duration to produce deepening of the channel occurred until the latter part of April, previous to which, as before stated, the works had deteriorated to such an extent that the outer 2 000 feet of the east jetty was 2 feet or more below mean gulf level. A large amount of the river discharge was consequently wasted laterally over the top of the jetties, but even with this loss of force a 15-foot channel was made across the bar, and the inner face cut away so that the 18-foot contours on opposite sides of the bar were only 400 feet apart. This channel has since increased in depth to 16 feet from the effect of the currents across the bar at a medium stage of the river. The bar in front of the west jetty was pushed seaward several hundred feet, but to the east of the axis of the jetty channel no change of position took place. A deposit of 1 to 2 feet of soft mud was made on the outer slope and beyond, which is now rapidly disappearing. It is quite evident

that if the jetties had been well above the water surface for their full length, the velocity of the current beyond the entrance would have been such that no deposit would have been made for a considerable distance in front of the works.

Under the plans adopted at the commencement of the work in 1889, it was the intention, as soon as the jetties were sufficiently settled, to build concrete coping walls on the portions exposed to wave action, of a similar construction to that designed and built by the writer at South Pass the previous winter.* The financial trouble of the past year prevented this work being done, and resulted in losses to the work fully equal in amount to one-half that which the walls would have cost, and which delayed the desired result of obtaining a deep water channel for at least one year. It is a notable fact that at South Pass previous to the construction of the concrete coping walls, a depth of only 24 feet was obtained, which in less than six months afterwards increased to over 30 feet. At the Sulina mouth of the Danube only 15 feet were maintained previous to consolidating the jetties with concrete blocks. This depth was increased to 22 feet by the natural forces of the stream very soon after both jetties were built to the same distance seaward and capped with a concrete superstructure. A similar result may be expected at the Brazos so soon as the jetties are thoroughly consolidated and strengthened with a proper superstructure. Until then they will be in constant danger of being wrecked by storms, and cannot be maintained in effective condition to give a much better channel than that now existing there.

Some of the more important principles governing jetty and harbor construction which became very apparent at different stages of this work may be worthy of note, and are briefly summarized below:

First.—Jetties, to produce a maximum result at a minimum expense must be completed beyond the bar in a single season if possible. The bar then acts as a submerged weir with the strongest current on the outer crest, thus transporting all eroded material to a safe distance from the entrance.

Second.—Delays cause great increase in the cost of construction from damage to the works by storms, and the much greater distance the jetties have to be built seaward.

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* For plates and description see *Railroad Gazette*, September 13th, 1889.

Third.—The success of jetty improvements depends largely on the existence of strong littoral currents in front of the harbor entrance, otherwise the advance of the foreshore and bar would soon close any channel obtained.

Fourth.—In jetties at the mouths of rivers, the strongest currents are at the outer end of the channel, and in no case is it necessary to build the works to a greater depth beyond the bar than that required in the channel.

Fifth.—When scour takes place in the channel bed its action is first noticeable at the lower end of the section and gradually works up stream.

Sixth.—In fresh water streams flowing directly into the gulf, salt water currents up stream often exist where the surface current is flowing seaward, and consequently surface velocities are no sure measure of the discharge or scouring force. When the east jetty was completed 2 000 feet in advance of the west, the main current at ebb tide flowed past the end of the unfinished jetty at nearly right angles to the jetty channel. The shoaling which then took place on the bar, and the subsequent deepening when the jetty was extended, very plainly indicate that one jetty would not be very effective for channel making at ports of this class.

Seventh.—To insure successful results for an enterprise of this character the engineer must not only have control of the plans for improvement and design of the structure, but also of the methods, material and rapidity of construction.

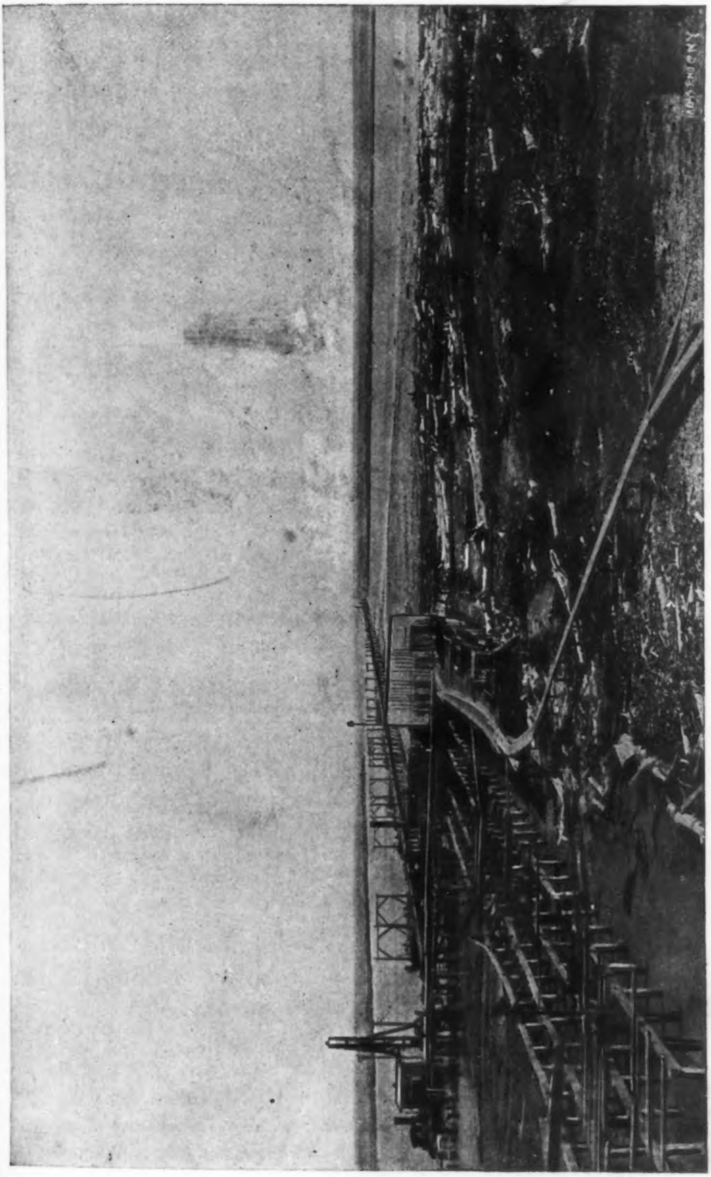
Description of Plates.

Plate XCVI. View of the pile-driver, trestle and suspended mats near the outer end of the west jetty.

Plate XCVII. View of the jetty channel from the head of the west jetty, showing the new shore-line to the west of the works, and in the left foreground, the mud deposit produced by wing-dams.

Plate XCVIII. View of the river looking down stream, near the upper side of the new town site. The width of the river at this point is about 500 feet, with a central depth of 23 feet. The wharves for the new town are being built along the concave bank shown on the left.

PLATE XCVI.
TRANS. AM. SOC. C. E.
VOL. XXV, No. 511.
WISNER ON BRAZOS RIVER HARBOR IMPROVEMENT.



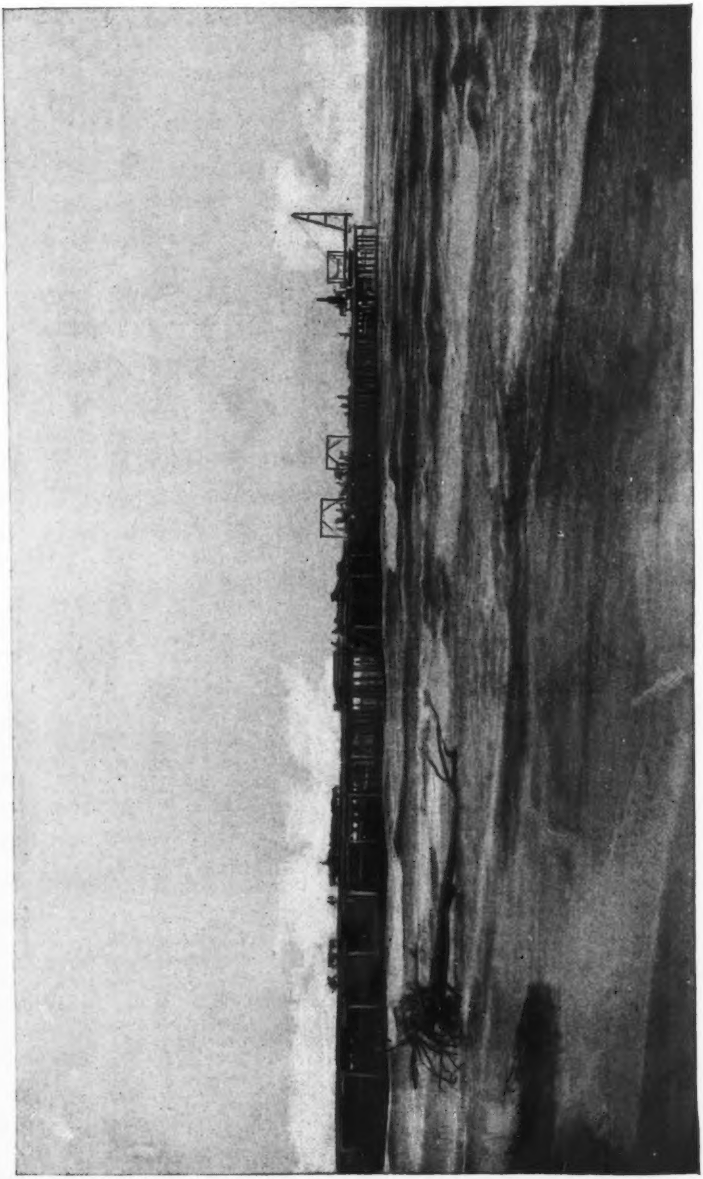
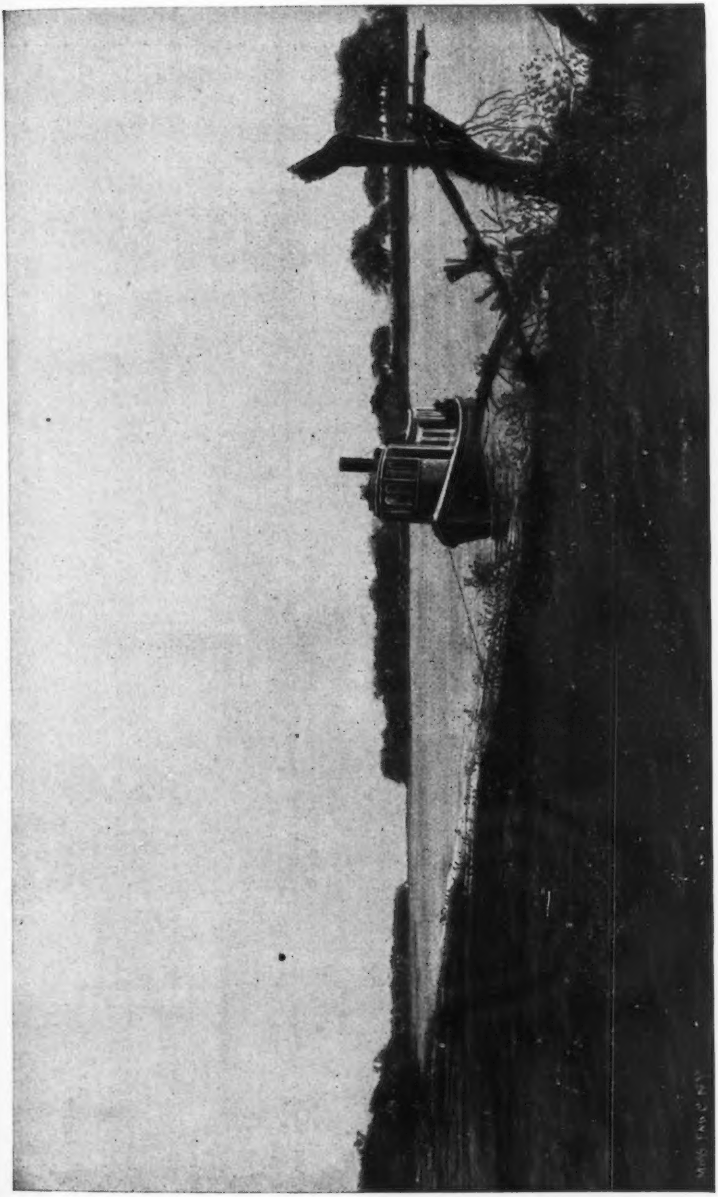
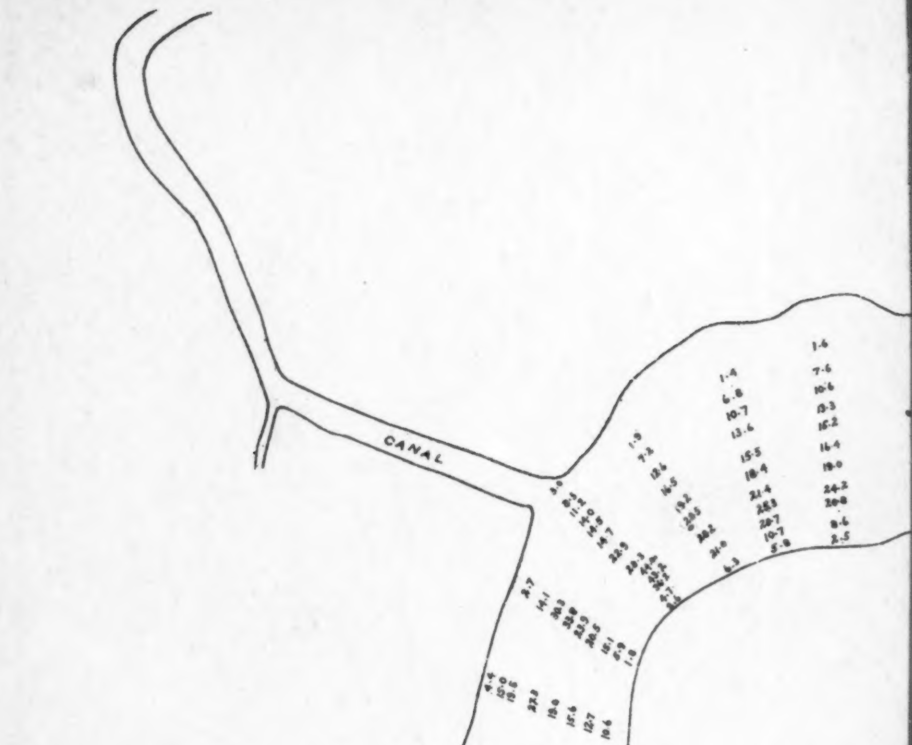


PLATE XCVIII.
TRANS. AM. SOC. C. E.
VOL. XXV, No. 511.
WISNER ON BRAZOS RIVER HARBOR IMPROVEMENT.

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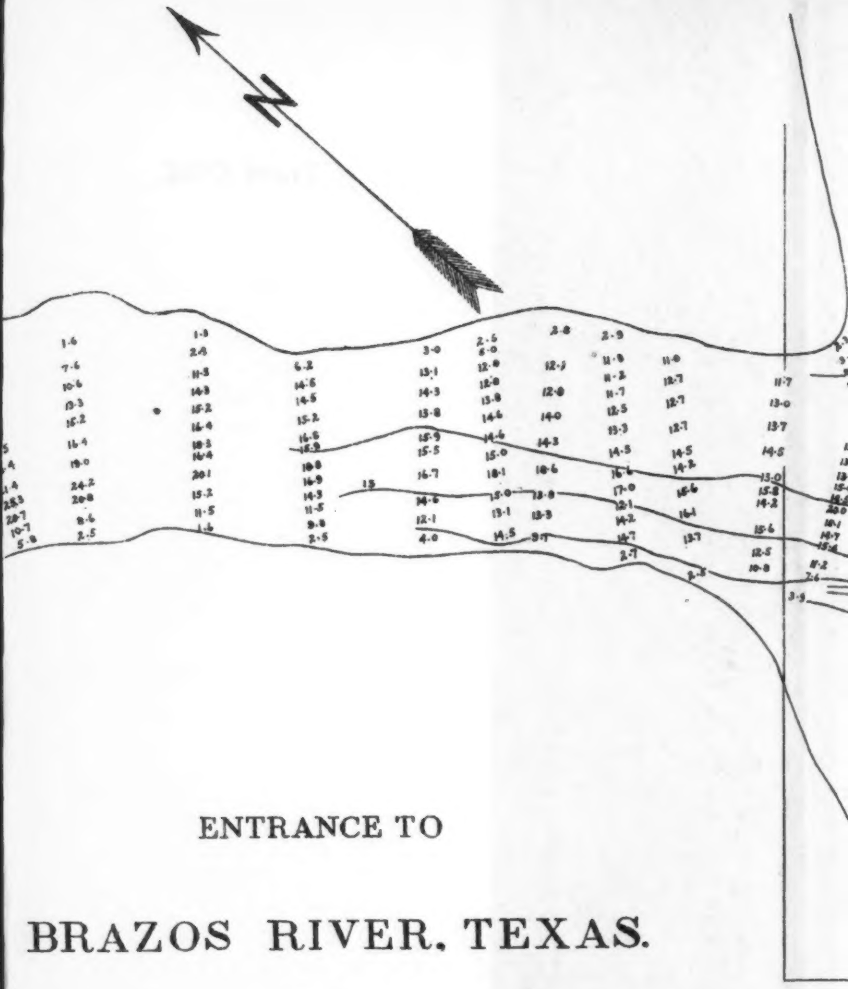

ENTRANCE TO

BRAZOS RIVER, TEXAS.

From SURVEY Made

Feb. & Mar. 1889

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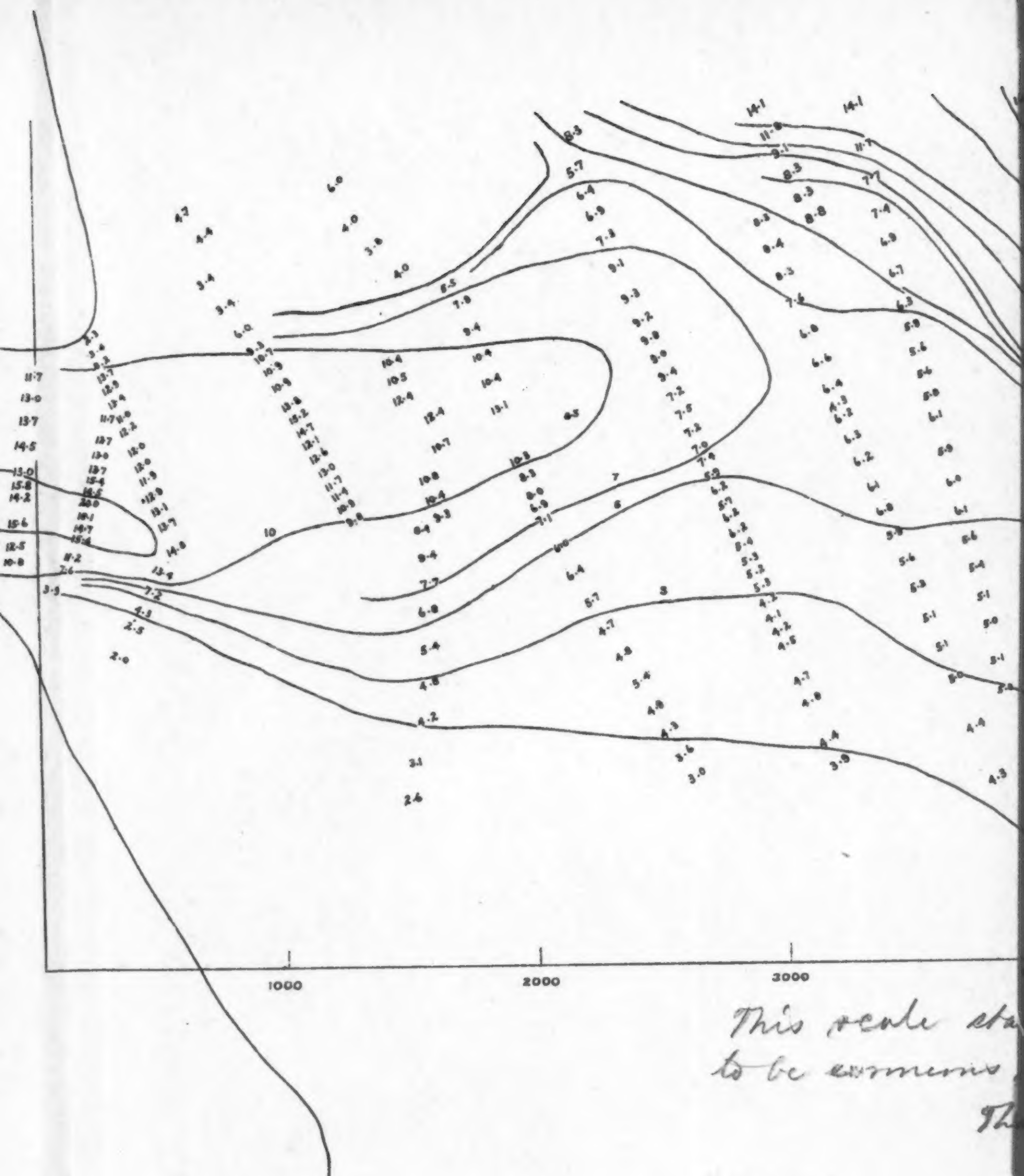

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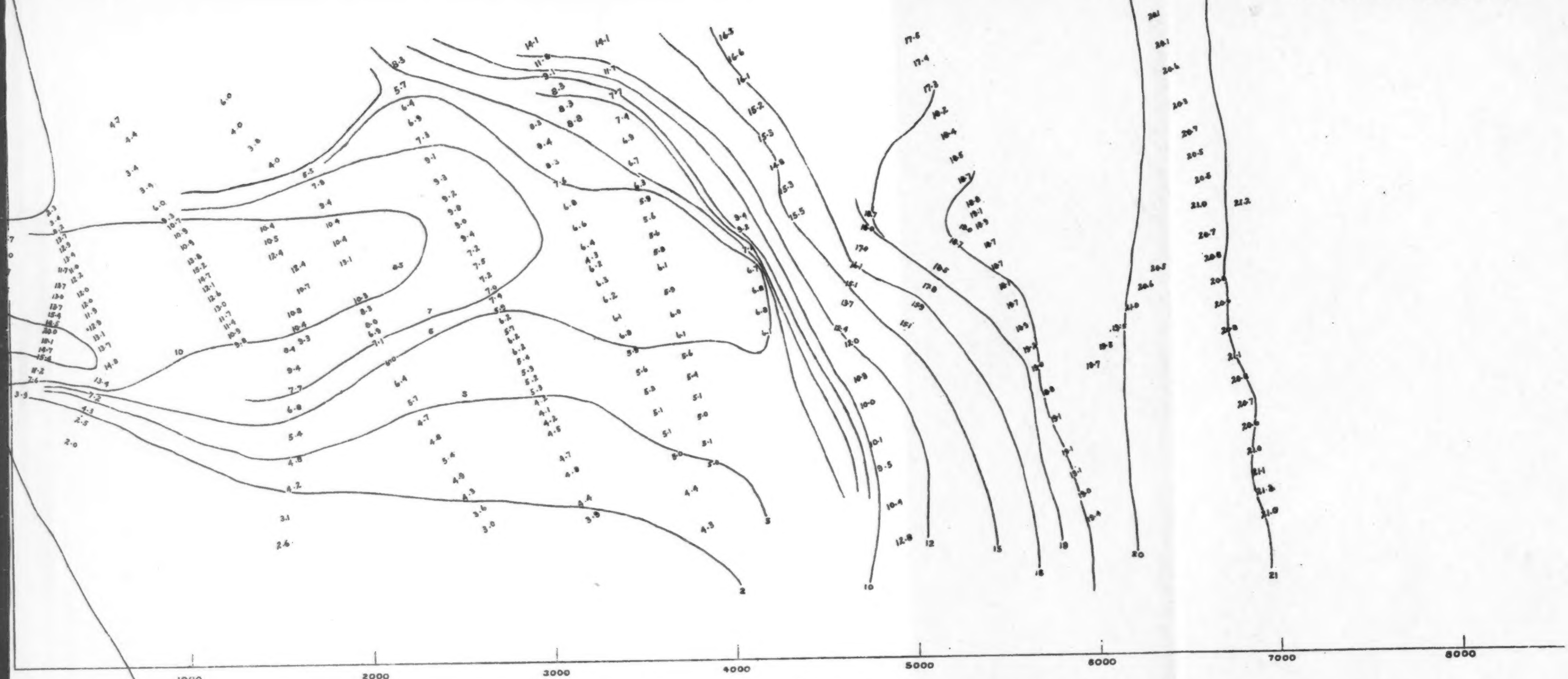

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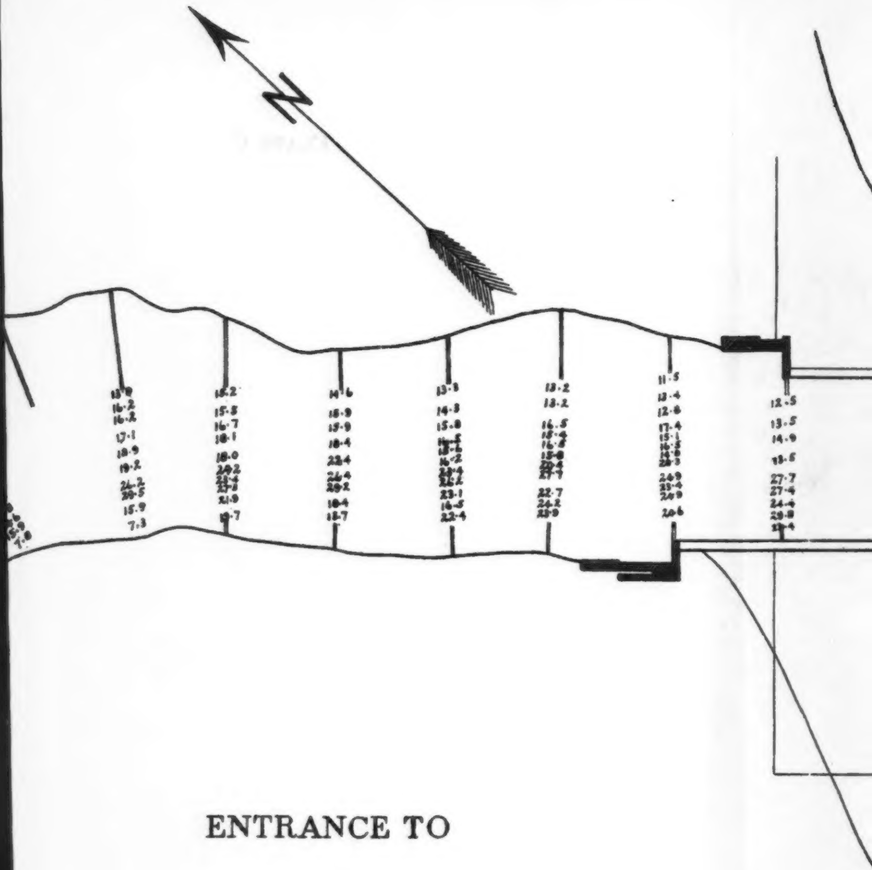


ENTRANCE TO

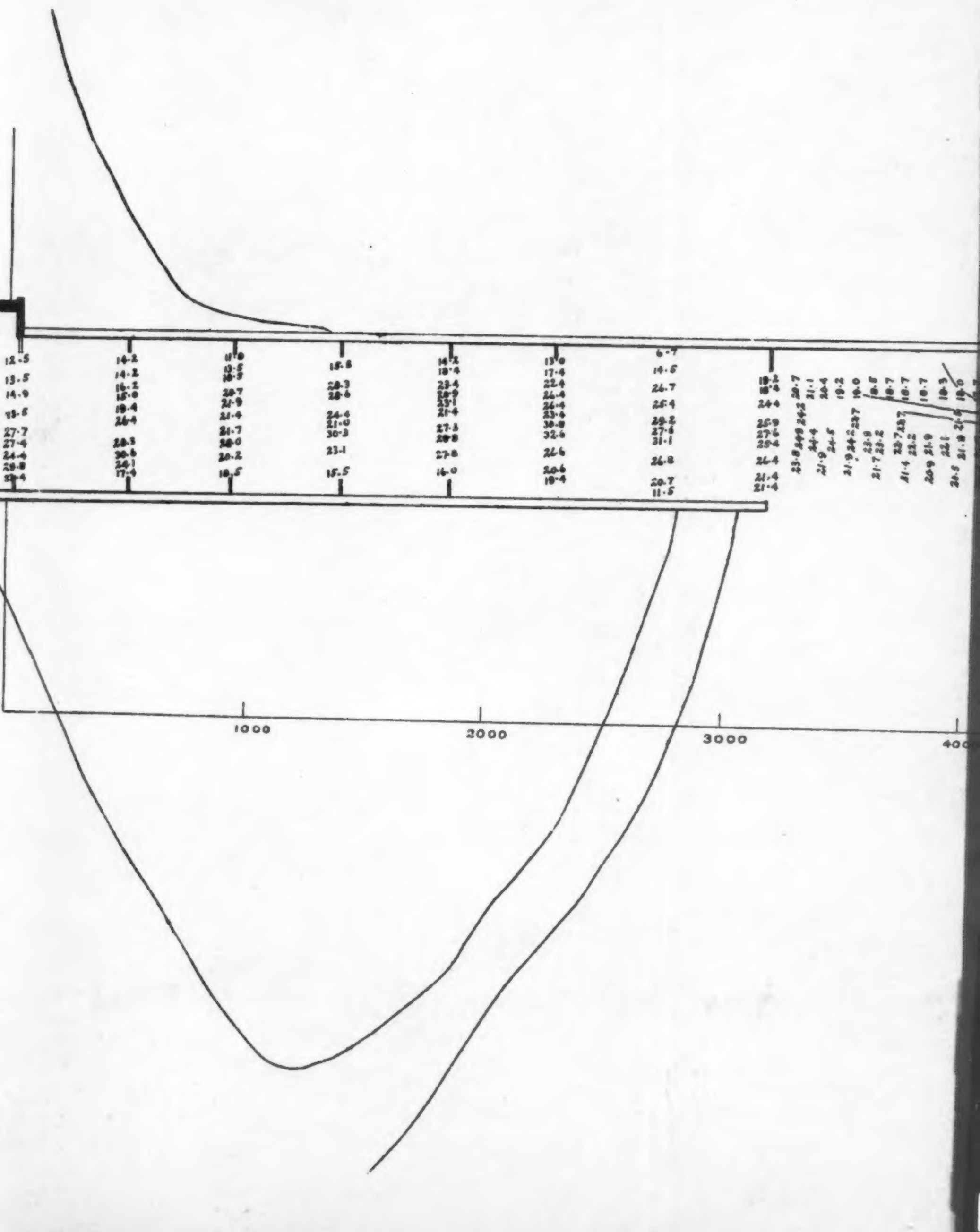
BRAZOS RIVER, TEXAS.

From SURVEY Made

Above Sta. 32 May 28, 1890.

Outside " " July 23, "

0 500 1000 FT. 2000 FT.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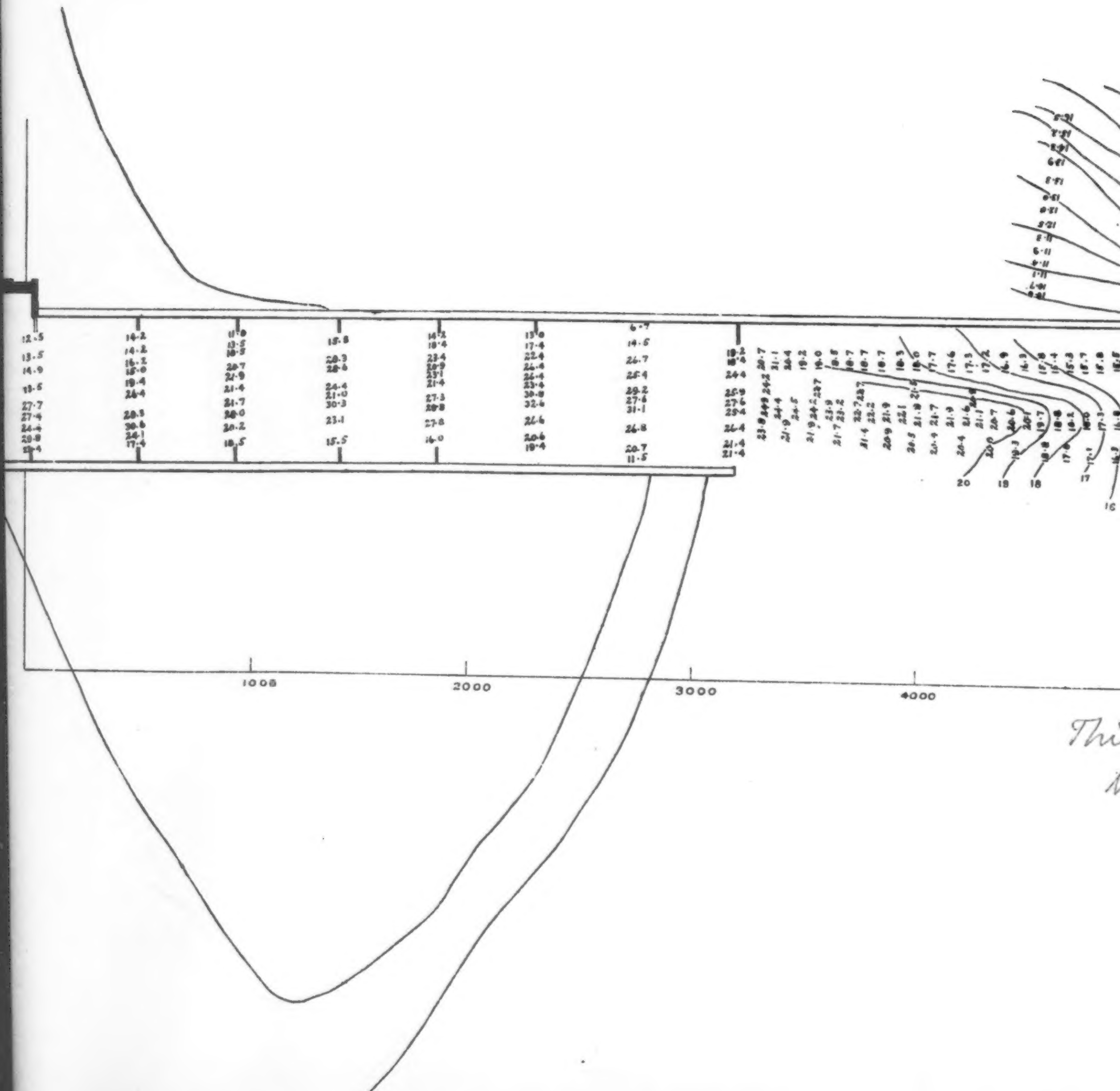
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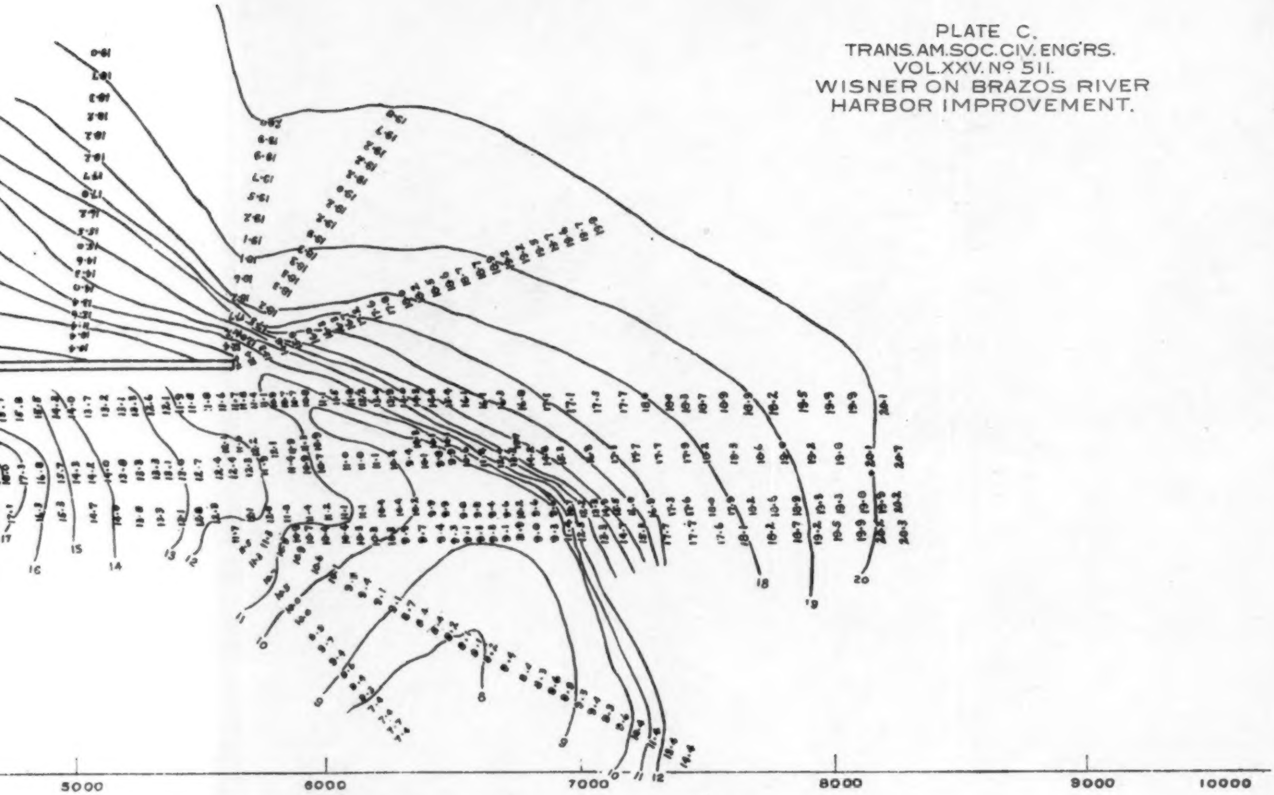
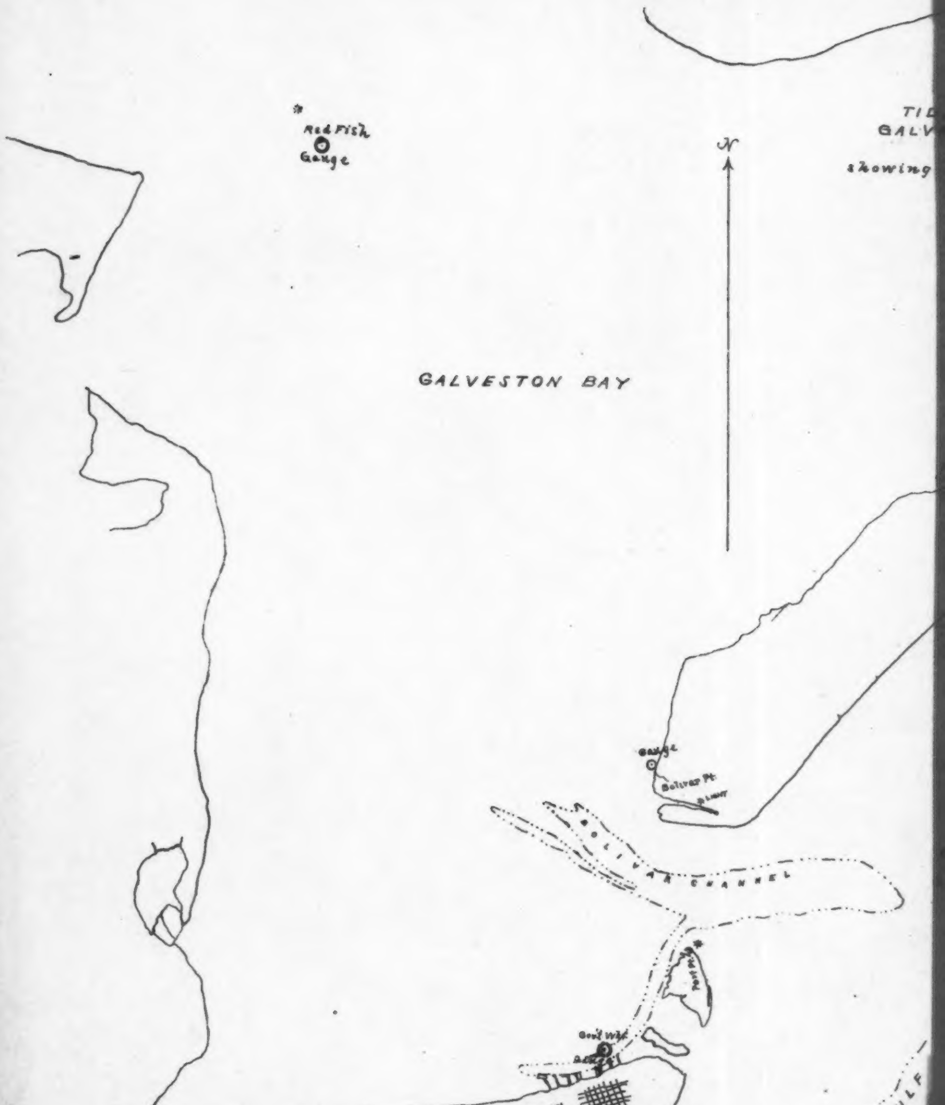
PLATE C.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV. Nº 511.
WISNER ON BRAZOS RIVER
HARBOR IMPROVEMENT.



This scale is stated in errata to
be erroneous, – i.e. about 12% too small,
That under title is correct.



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GALVESTON
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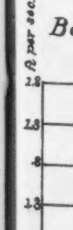
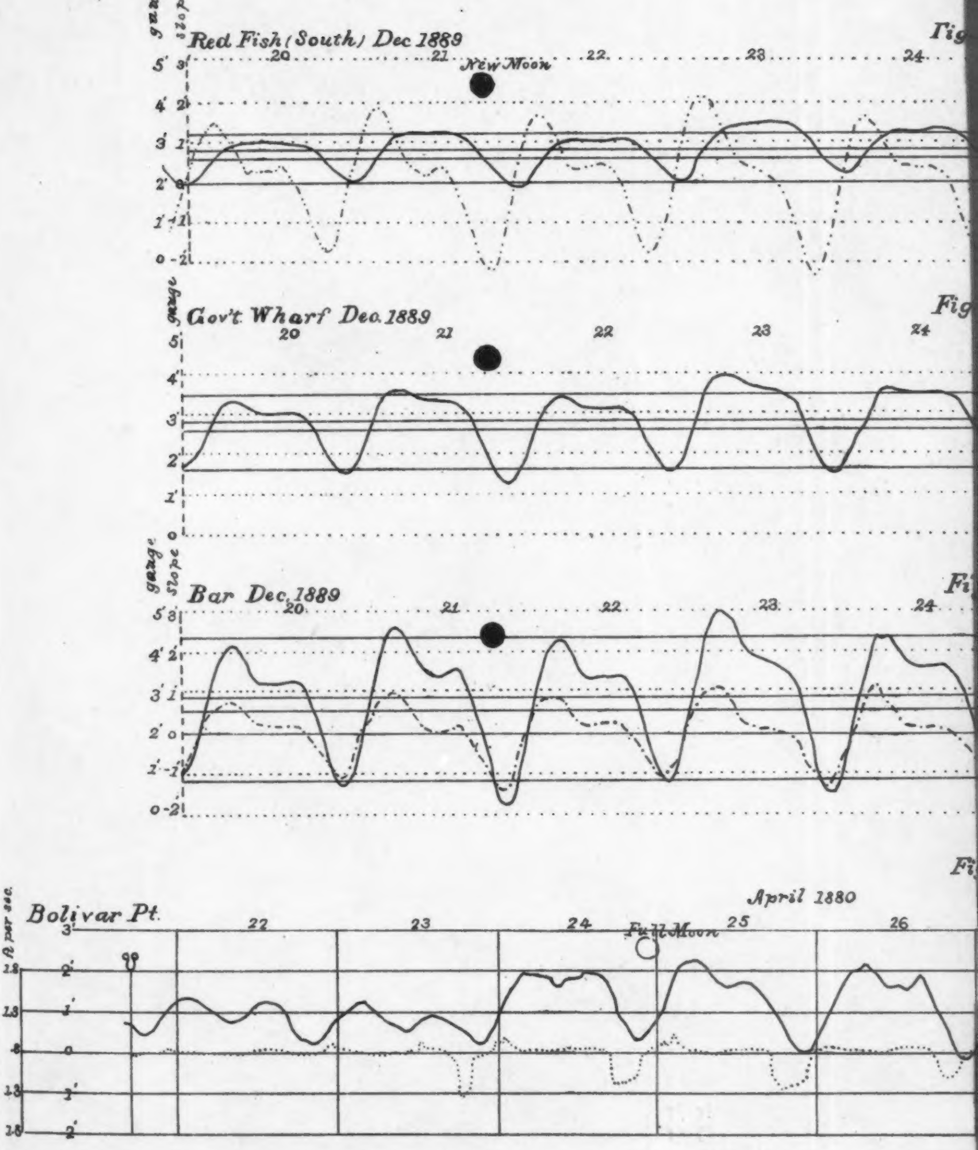
PLATE CI.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV. Nº 511.
RIPLEY ON TIDAL PHENOMENA
IN GALVESTON HARBOR.

B



Tidal Phenomena, C



nomena, Galveston Harbor

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Fig. 1

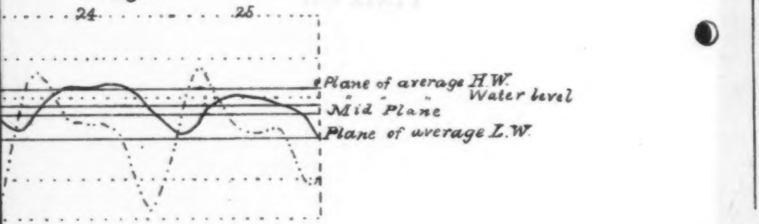


Fig. 2

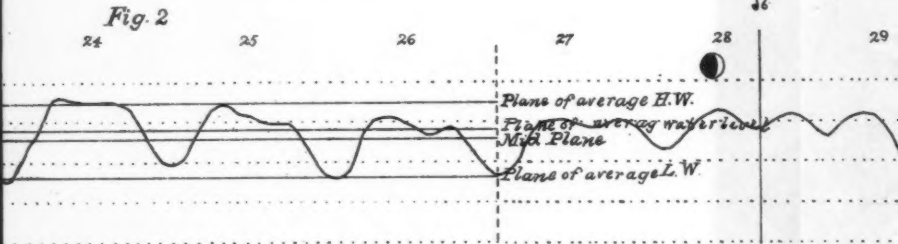


Fig. 3

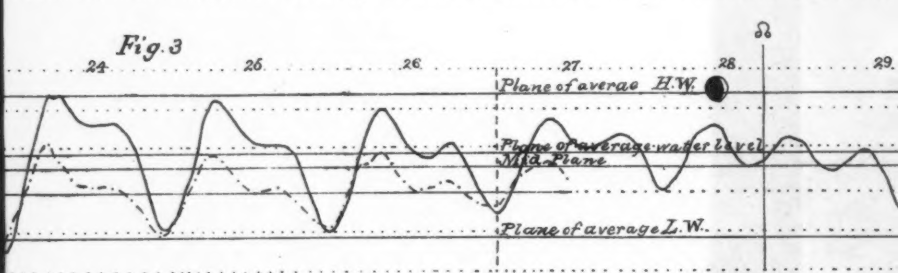


Fig. 4

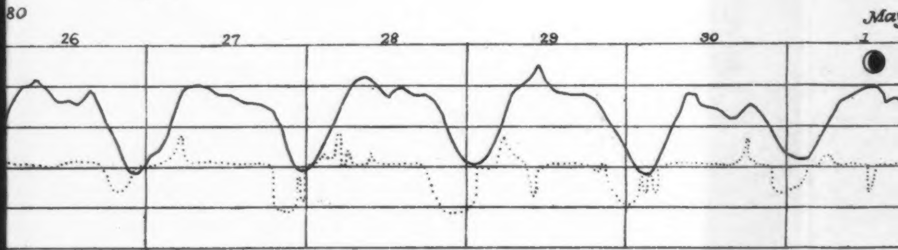


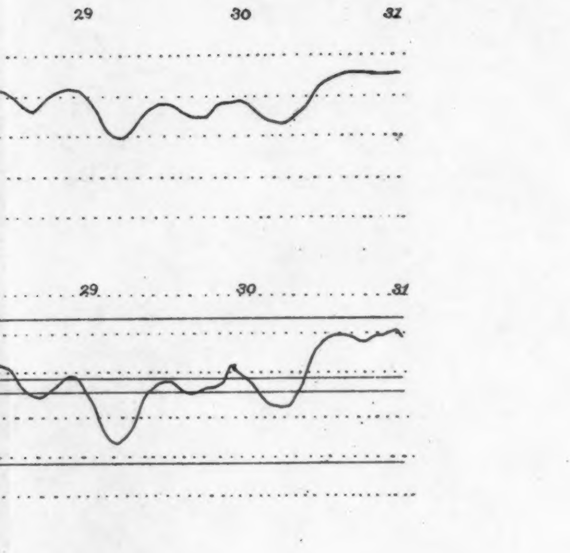
PLATE CII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXV. № 511.
RIPLEY ON TIDAL PHENOMENA
IN GALVESTON HARBOR.

Note

Tidal curves are shown thus

Slope curves { Bar to R. F. . / " " Gov't Whf. "

Velocity curve is shown thus



ERRATA.

By an exchange of tracings in preparing the matter for publication, the long scale to the right on plates XCIX and C in the November number, page 537, was erroneously inserted.

The *correct* scale for each is that given under the title. The other is about 12 per cent. too small.

The numbers on plates XCVI and XCVII should be reversed.

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Plate XCIX. Chart showing the depth of channel in the mouth of the river and on the bar just previous to beginning the jetty construction.

Plate C. Chart showing the improved river channel above the jetties, the extension of the bar seaward caused by the freshet of May, 1890, and the position of the jetties at the time of the subsequent shoaling of the channel. The stations are 100 feet apart and number from the head of the west jetty. Six months after the date of this survey, erosion had taken place to such an extent that the 20-foot curve was only 100 feet in front of the jetty entrance.

DISCUSSION.

G. H. Mendell, M. Am. Soc. C. E.—It appears from official reports that depths upon the bar at Brazos River, in its natural state, have varied from 2.5 feet to 8.5 feet; that changes have been very sudden and of constant recurrence; that in 1887 the maximum draft of vessels trading there was 4½ feet; that during the previous September no vessel was able to cross for want of depth; and that the magnitude of the bar has varied greatly, as evidenced by the positions of the interior and exterior 12-foot contours—the interval having been found as follows: in 1858, 2 100 feet; in 1875, 4 650 feet; in 1881, 1 750 feet; in 1882, 1 200 feet, and in 1887, 4 500 feet. The alignment of the bar channel has also varied very much. After a freshet it is direct. Subsequently it undergoes frequent changes, being at times perpendicular to a former position.

There is material discrepancy between the description of the lower portion of the river, as given in official reports, and in the author's account. But both so far as they relate to depth are very general; one stating "about 12 feet with pools of 18 feet," the other "15 to 50 feet."

It appears from the paper that the following conditions exist, namely; almost an absence of tide which, however, must supply most of the low water discharge, reported to be as little as 1 000 feet per second; a duration of the low water season from one to three months; freshets of short duration carrying 60 000 cubic feet or more per second with one-two hundred and fiftieth of their weight in sediment, estimated for the year at 10 000 000 tons, a very extraordinary variation in flow, the maximum

being of short duration and sixty-fold the minimum and even more, a current to the westward 1 to 3 miles an hour due to winds, an unfavorable action on the bar during the prevalence of summer winds from an opposite direction; a low slope of the bed of the ocean, namely, 1 foot in 500, great destructiveness by the teredo, costliness of stone involving construction of jetties mainly in brush. The resultant of these conditions thus baldly stated cannot be considered as altogether promising. If they could be defined in their relative values, favorable and unfavorable, perhaps an *à priori* judgment of a definite character might properly be made as to the measure of success likely to attend this improvement. But this is not now possible.

The circumstances of an uncompleted work, while interesting and suggestive, perhaps, cannot carry the lessons which are to be derived from consideration of the conditions found to exist several years after completion. At this later period we shall be informed whether or not the recurrence of former conditions, so generally appertaining to bar improvements of non-tidal rivers, shall here obtain.

If the jetties prove to be permanent, escaping the destructive action of the teredo, the effects of the storms of the Gulf, and other hostile agencies that may exist, we shall always expect to find between their lines a sufficient depth, and if the exterior forces working for good shall, year by year, sufficiently remove from the front of the jetties the 10 000 000 tons of silt brought by the river and the sand drifted from the east, there may be 20 feet of water over the bar, as is expected by the engineer. But if the westward movement at the bar be unable to take care of the material presented to it, whatsoever be the depth obtained by a speedy construction of jetties, we may expect, after an interval of time proportioned to the resultant amount of deposit, that something like the old entrance will be restored a few thousand feet in front of the end of the jetties. This interval of time would, of course, be greater, other things being equal, if the slope of the ocean bed were greater. The final issue appears to be rather in the hands of nature than within control of man.

It is not important, but it may save misconception, to say that the method of construction of mats designed by the engineer of this work in 1888, viz., by suspension from the timbers of the trestle, stated by him to have been successfully used by United States engineers on the Pacific Coast for the past two years, was first applied here on the harbor improvement at Yaquina Bay in the spring of 1883. It has been continuously in use by the United States engineers for the past eight years at Yaquina, and at the mouth of the Columbia River. The method is mentioned in the Chief of Engineer's report, 1883, page 2066, also in 1884, page 2265.

O. H. Ernst, M. Am. Soc. C. E.—The questions involved at the mouth of the Brazos are simply these: how large a channel will a river confined

by jetties carve for itself, the discharge of which varies from over 60000 to less than 800 cubic feet per second; and what portion of the maximum channel secured during the floods can be maintained during low water, in the face of wave-action upon a low and sandy sea-coast? The engineers who attempted to answer these questions by theoretical discussion obtained widely different results. According to some there would be no difficulty in making and maintaining a channel 20 feet deep and 100 wide, or even a greater channel. According to others a channel 10 feet deep and 100 feet wide could be obtained with jetties 700 feet apart, and a greater depth for that width with jetties closer together, but it would be impracticable with any degree of contraction to maintain a depth of 20 feet and width of 100 feet—the least dimensions at all suitable for a naval station or harbor of refuge—and it was even possible that under a certain combination of circumstances, not uncommon to the locality, the dimensions might be reduced to 6 x 100 feet, that is that there would be no improvement at all. Somewhere between these two extremes there might be some permanent improvement, but the degree of it was so uncertain that the whole enterprise must be pronounced a costly and doubtful experiment. The result of the practical demonstration by the actual construction of jetties which has been undertaken, has therefore been, and is still, awaited by them with much interest.

Further theoretical discussion at this time would seem to be profitless, but all recent and detailed information about the progress being made in improving the channel is interesting. It is to be regretted that the only map published with this paper is from a survey made more than a year ago, and that the width of the 16-foot channel obtained during the freshets of two or three months ago is not given. It is even difficult to ascertain with accuracy the clear width of waterway between the jetties. At one place in the text the distance between the jetties is mentioned as 600 feet, but subsequently reference is made to a change of plan. Scaled from the map the distance is about 560 feet, reduced by the spurs to about 450 feet. A depth of 16 feet, or a much greater "central depth, without regard to width," proves nothing for or against the correctness of either of the opposing opinions. The dimensions of channel which can be maintained throughout the year and every year, are what mark the degree of improvement of which the locality is capable. The season of low river and heavy cyclonic disturbances of the gulf is just approaching. It will be interesting to know what effect these circumstances will have had a few months hence, whether there will have been any decrease in the dimensions of the channel, and if not whether such changes will have occurred in the shore-line as will render such decrease more or less probable upon the recurrence of a similar season. It is to be hoped that the engineers familiar with the work will inform the Society at that time, and will furnish a map showing the hydrography to date.

The operation of constructing and placing the mattresses was ingenious and possessed some novel features. Inshore the use of that material appears to have been successful. Whether it will prove to be so at the sea ends of the jetties, remains to be seen. The stripping by heavy seas of the ballast from the top of the east jetty, which was in pieces weighing only from 1 000 to 3 000 pounds, was to be expected. Much heavier blocks are needed here. The paper contains many constructional details, for which the author merits the thanks of the Society. The general question, however, of the practicability of making a permanent channel 20 feet deep and 100 feet wide, is where it was before the work began. Nothing has occurred as yet to controvert the opinion expressed in the report which caused the Government to abandon the improvement.

It would, perhaps, have been better if the author had deferred the publication of his paper until something definite had been obtained from the practical test.

ARTHUR J. MASON, M. Am. Soc. C. E.—Mr. Wisner's paper on the Brazos River improvement, brings out strongly the value of littoral currents in aiding the improvement of channels at harbor entrances. The function of the channel current when contracted, is to loosen and hold in suspension the solid matter, the function of the littoral current to transport that material out of the way, in common parlance, to waste or spoil it. In a recent visit to the mouth of the Brazos, the part played by the littoral current was very strikingly shown by that lightest of all silt, drift wood, the angle formed by the west jetty and the west shore being covered for perhaps a hundred acres with a labyrinth of tree trunks brought down by the river, the beach eastward showing very few.

The success of the work at the mouth of the Brazos, and at other places, shows that the actions of the sea currents and other forces are no longer to be regarded by engineers as whims of nature, too complicated, too arbitrary, too intangible for human observation; that these forces are capable of subtle, careful analysis, based on close, long extended observation, that the treatment of harbor channels can be undertaken with the same confidence and success as the many equally great enterprises of which one hardly hears, but that they are proposed, under contract, and successfully completed.

LEWIS M. HAUPT, M. Am. Soc. C. E.—The question of securing a deep water port on the Texas coast is one of great interest to the entire country. While engineer officer of that district after the war, I was impressed with its importance, and subsequently I have given considerable attention to the various projects and efforts which have been made to this end. In this paper, Mr. Wisner has given a carefully prepared résumé of the history and operations at the mouth of the Brazos River, showing very fully the attempts and failures, and stating the reasons therefor, thus making it an instructive example for the use of maritime engineers. Much credit is due the syndicate for the courage exhibited in undertaking a project which had just been so entirely condemned as

impracticable, but it must have been evident to them, as to others, that a correct diagnosis had not been made of the physical conditions, and that the case was not therefore altogether hopeless.

Of the several inlets which indent this coast, the Brazos is the only one between Sabine and Corpus Christi which is not dependent entirely upon the tidal volume for its action. Having no interior lagoon or estuary, it belongs to the class of rivers which debouches directly into tidal water, and which, therefore, offers the most favorable conditions for improvement by parallel jetties properly spaced and located.

With reference to this first point, viz., width between jetties, the interval is admirably adapted to secure the best results for this particular river, but, as to the location, it is as yet incomplete and subject to change as to length, according to varying conditions. The original length has, I believe, been already exceeded in consequence of unforeseen delays in the prosecution of the work. The direction was selected with reference to the prevailing seas, requirements of navigation, discharge of fresh water and the littoral currents, and seems to be a happy compromise between these somewhat conflicting conditions. The case is peculiar in its physical conditions and in its resemblance to the few successful instances of jetties in pairs at the mouths of rivers with weak tides, but where there is a large tidal movement, even though the range be small, and especially where there is a prevailing direction of littoral drift.

I have advocated for some years the construction of a single line of defensive works to protect the channel on the outer bar of inlets from the traveling sands, as being a better expedient for tidal entrances than the usual pair of jetties intended to produce concentration of ebb currents, and I believe that the more recent practice has fully confirmed the correctness of these principles. One of the latest instances of this kind is to be found at the mouth of the Columbia, in Oregon, where the single jetty which has been built on the south side of the entrance has intercepted the drift of sand and shingle and thus improved the depth several feet, although the bar has been pushed seaward and is still in an unstable condition, because of the improper form and location of this work. The original project contemplated another short jetty from Cape Disappointment to increase the contraction. This should not be built, as it would still further increase the shoaling which has taken place inside of the capes and reduce the tidal prism entering the estuary.

During a recent visit to the Pacific coast harbors I have learned of another instance of the failure of the twin jetty plans as designed for Coos Bay, Oregon, where the entrance has been so obstructed that the vessels which formerly sought shelter there now avoid it entirely as being too dangerous.*

* This is on the authority of one of the largest shippers of the Pacific coast.

Of the attempt made to obtain deep water at Galveston by twin jetties located 7 000 feet apart, I need only say that under the present plan it must result in a calamitous failure, as such jetties cannot co-act to produce scour, and are too remote from each other to protect the channel.* Even those at Sabine Pass are admittedly too far apart to produce the desired result. While the other inlets at Corpus Christi, Cavallo and Aransas have been abandoned as incapable or unworthy of improvement; and yet it is possible with a comparatively small scour to create a good channel over the bar.

The results thus far attained at the Brazos River are evidence of the success which may be expected to attend all well designed and rapidly executed projects of this character, and the projectors are to be congratulated upon the success which has thus far attended their efforts in the face of many discouragements and difficulties.

Mr. H. C. Ripley.—The admirable paper of Mr. Wisner on the Brazos River improvement will certainly be read with the greatest interest by all engineers in this country who have given any study to this branch of engineering; for, without doubt, the success achieved at the Brazos is the most signal of its character since the accomplishment of the South Pass improvement. Our national legislators will also be interested in this paper if they reflect that this work has been accomplished by private enterprise under exceedingly embarrassing financial conditions within three years of the time of the passage of the bill in Congress authorizing the company to commence operations there.

Mr. Wisner's paper gives the impression that no guesswork was permitted to usurp the place of exact information regarding the essential features of the problem to be solved; and, having this information, the engineers were able to express unlimited confidence in the success of their work.

A board of engineers had previously expressed its opinion concerning a plan which it had devised for the improvement of an important harbor in these words: "It will be seen that the board does not attempt any prediction of the precise depth the jetties will maintain. Such predictions can best be made by those ignorant of experience in tidal entrances elsewhere, and having great confidence in the credulity of mankind."

It is unfortunate that the author did not furnish with his paper a chart of the last survey of the Brazos entrance. During a recent visit there the writer had an opportunity of inspecting this chart. A comparison of it with those of the Sulina mouth of the Danube and South Pass, Mississippi River, shows a similarity almost as great as that which charts of two different surveys of the same place would show. In each case we have a sediment-bearing stream of fresh water debouch-

* See Franklin Institute Journal, October, 1891, for a brief history of this work, with statement of results to date.

ing into a sea of salt water, a littoral current flowing past the mouth in one direction and a similar treatment by jetties extending normal or nearly so to the direction of the littoral current. And we find in each case the deep water close to the end of the jetty on the side from which the littoral current comes, an immense shoal extending far out beyond the end of the jetty and a new shore line formed far out beyond the original one on the side opposite to that from which the littoral current comes, showing that the laws which control the currents and waves are as unerring in their action as the laws which control the heavenly bodies. To predict results, therefore, where the controlling forces are known is no more hazardous for the engineer with experience in such matters than for the astronomer to predict an eclipse.

A comparison of the last chart of the Brazos with one of the Sulina mouth of the Danube at a similar stage of progress of the jetties there, furnishes a strong inductive proof that when the Brazos jetties shall have been built up throughout and consolidated and strengthened so as to effectually resist the action of storms, there will be no more uncertainty about the permanency of the depth there than there is about the permanency of the depth at the Sulina mouth of the Danube or at South Pass.

The author's general conclusions give evidence of his having given the subject of harbor improvements in general a careful and intelligent study. These remarks on the improvement of tidal harbors, however, would seem to require some modification to make them applicable to all cases. He says:

"With tidal harbors, however, the slope of the surface in the jetty channel is inversely as the length of the pass; and consequently on the Gulf where the tides are very small (about 1 foot) and the distance to deep water very great, the plan of improving harbor entrances by confining tidal currents between jetties is a somewhat doubtful experiment."

It is admitted that such a plan is an expensive and unscientific method of improving tidal harbors, inasmuch as the improvement which can be effected by such means, where possible, will be only a fraction of what is possible by other means, and where the form of the tidal curve is that of a sinusoid like that on the Atlantic coast, the experiment is exceedingly doubtful. However, where the form of the tidal curve is such as is developed on the Texas coast during great declinational tides, it is possible by confining tidal currents between jetties to greatly increase the water slope, and hence the scouring force due to tidal action over its normal amount, especially during ebb currents. The results of an investigation made more than a year ago by the writer on water slopes and other tidal phenomena, seem to sustain the position here taken. They are given below, and are entitled—

TIDAL PHENOMENA OF GALVESTON HARBOR.

"For the purpose of investigation we have selected the period of great declinational tides extending from December 20th to 27th, 1889.

This was a time of freedom from wind disturbance. It was also a time of conjunction of the sun and moon, and hence at a time when large fluctuations should be expected. In Fig. 1, Plate CII is given the tidal curve at Red Fish (south side) 17 miles from Galveston Bar and the curve of slope between the bar and Red Fish. Fig. 2 gives the tidal curve at Government wharf, Galveston, 6¼ miles from the bar. Fig. 3 gives the tidal curve at Galveston Bar and the slope curve from the bar to the Government wharf. For the period considered the plane of average level of the water is shown by a full line. The plane of average high water, average low water and the plane midway between them are also shown

TABLE No. 1.

TIDAL PHENOMENA, GALVESTON HARBOR, DECEMBER 20TH TO 27TH, 1889.

Averages.	Bar. Feet.	Government Wharf. Feet.	Red Fish. Feet.
Water level	2.90	2.87	2.84
Mid plane	2.57	2.64	2.61
Difference	0.33	0.23	0.23
High water	4.41	3.59	3.23
Low water	0.74	1.69	1.99
Fluctuation	3.67	1.90	1.24

by full lines. Table No. 1 gives the results shown on the drawing, in figures, and also the fluctuations. It was constructed as follows:

The average water level was determined from the plotted curves by means of the planimeter. The slight difference obtained at the different points is probably due to an error in the plane of reference which was determined by water level comparison. The other quantities in the table were determined in the usual way.

TABLE No. 2.

TIDAL PHENOMENA, GALVESTON HARBOR. WATER SLOPES BETWEEN GOVERNMENT WHARF AND BAR, DECEMBER 20TH TO 27TH, 1889.

DATES.	EBB.			FLOOD.			Ratio of Ave. Ebb to Ave. Flood.
	Duration. Hours.	Average. Feet.	Max. Feet.	Duration. Hours.	Average. Feet.	Max. Feet.	
20th and 21st	8	0.08	0.90	16	0.38	0.80	1.8
21st and 22d.	10	0.75	1.15	15	0.45	1.00	1.7
22d and 23d..	8	0.73	1.50	16	0.42	.90	1.8
23d and 24th.	11	0.73	1.20	14	0.52	1.10	1.4
24th and 25th	11	0.63	1.45	14	0.41	1.15	1.5
25th and 26th	11	0.50	1.25	9	0.47	0.85	1.1
26th and 27th	8	0.31	1.05	15	0.36	0.90	.9
Totals	67	4.33	8.50	99	3.01	6.70	10.2
Means	9.4	0.62	1.21	14	0.43	.96	1.5

The ratio of flood to ebb duration is $\frac{14}{9.4} = 1.5$.

TABLE No. 3.

TIDAL PHENOMENA, GALVESTON HARBOR. WATER SLOPES BETWEEN BAR AND RED FISH SOUTH, DECEMBER 20TH TO 26TH, 1889.

DATES.	EBB.			FLOOD.			Ratio of Ave. Ebb to Ave. Flood.
	Duration. Hours.	Average. Feet.	Max. Feet.	Duration. Hours.	Average. Feet.	Max. Feet.	
20th.........	3	0.75*	....	....	....	1.55	
21st..........	9	0.99	1.80	16	0.68	1.90	1.4
22d..........	10	1.23	2.35	15	0.85	1.70	1.4
23d..........	8	1.19	1.85	16	0.78	2.20	1.5
24th..........	10	1.32	2.45	16	0.98	1.45	1.3
25th..........	9	1.02	1.85	16	0.74	1.75	1.4
26th..........	6	0.87*	1.25	16	0.65	....	1.3
Totals........	55	6.48	11.55	95	4.68	10.55	8.3
Means........	$9\frac{1}{6}$	1.08	1.92	15.8	0.78	1.76	1.4

* These are combined and give slope of 0.83 for 9 hours.

The ratio of flood to ebb duration is $\frac{95}{55} = 1.73$.

Tables Nos. 2 and 3 give the average slopes and corresponding durations, together with the maximum slopes and the ratio of the flood slopes to the ebb slopes. The average ratio of duration of flood slope to ebb slope is also given and the maximum ebb and flood slopes. For example: in Table No. 2, December 20th and 21st, the ebb slope between the Government wharf and bar continued for eight hours, the average of which taken at hourly intervals was 0.68 of a foot. The flood slope continued for sixteen hours with an average of 0.38 of a foot and the ratio of ebb to flood of 1.8 indicates that the average ebb slope exceeded the average flood slope by 80 per cent.

By a further inspection of Table No. 2 it will be seen that the average duration of ebb slope between the bar and Government wharf is 9.4 hours, the average ebb slope is .62 of a foot, the average duration of flood slope is fourteen hours and the flood slope is .43 of a foot. The ratio of the ebb slope to the flood slope is $\frac{.62}{.43} = 1.5$. In other words, the average ebb slope exceeds the average flood slope by 50 per cent. Similarly we find the ratio of the duration of flood to ebb $= \frac{14}{9.4} = 1.5$. In other words, the duration of the flood slope is 50 per cent. greater than that of ebb slope.

The reason for these phenomena is apparent from an inspection of the form of the tidal curve. The longer duration of high water shows that

the average level of the water is above the plane midway between the planes of mean high and mean low water, and hence the distance from average water level to low water is greater than to high water. These facts are shown in their relative values in Table No. 1, where it will be seen that the plane of average water level at Government wharf is 2.87 feet above an assumed datum, while the mid plane is but 2.64 feet above the same datum, giving a difference of .23 of a foot. This difference is .33 of a foot at the bar, and .23 of a foot at Red Fish. It will also be seen that the plane of average low water at the bar is 2.13 feet below the plane of average level at Government wharf and the plane of average high water at the bar is 1.54 feet above the plane of average level at Government wharf, giving an average slope of .59 of a foot in favor of the ebb, or a difference of 38 per cent. in favor of the ebb.

These quantities are obtained as follows: $2.87 - 0.74 = 2.13$; $4.41 - 2.87 = 1.54$; $2.13 - 1.54 = 0.59$; $\frac{.59}{1.54} = .38 = 38$ per cent.

Referring again to Table No. 2, it will be seen that the average of the maximum ebb slopes is 1.21 feet, while that for flood slopes is .96 of a foot—an excess of 26 per cent. in favor of the ebb. Also the maximum ebb slope is 1.50 feet, while the maximum flood slope is 1.15—an excess of 30½ per cent. in favor of the ebb. It will thus be seen that the mean ebb slope, the mean maximum ebb slope and the maximum ebb slope are greater than the corresponding flood slopes, and by amounts of 50 per cent., 26 per cent. and 30½ per cent. respectively.

Whether the ebb and flood velocities bear the same relation to each other as the slopes, may be a matter of some interest, although as far as actual scouring force is concerned, that force depends quite as much if not more upon water slope than upon velocity. This subject, however, has received some attention, and in Fig. 4, Plate CII is shown a record of velocity observations obtained at Bolivar Point in 1880, by means of an automatic current meter described in the report of the Chief of Engineers, 1880, page 1220. This velocity curve indicates that the difference in ebb and flood velocities is even greater than the difference in slopes, and a velocity record obtained at a point immediately upon the bar for a shorter period of time indicates the same fact. Nor should such a result be unexpected, for by an examination of the tidal and slope curves it will be seen that to inaugurate a flood current requires that the momentum due to a strong ebb current and the inertia of the water in the bay be overcome, while to inaugurate an ebb current requires only that the momentum due to a very gentle current and the inertia of the water be overcome. We thus see that the ratio of ebb to flood velocities is still greater than the ratio of their slopes.

TABLE No. 4.

TIDAL PHENOMENA, GALVESTON HARBOR. WATER SLOPES BETWEEN GOVERNMENT WHARF AND BAR, ASSUMING WATER IN BAY TO REMAIN AT AVERAGE WATER LEVEL, DECEMBER 20TH TO 27TH, 1889.

DATES.	EBB.			FLOOD.		
	Duration. Hours.	Average. Feet.	Max. Feet.	Duration. Hours.	Average. Feet.	Max. Feet.
20th and 21st	9	1.38	2.17	14	0.31	0.53
21st and 22d.	9	1.75	2.82	15	0.95	1.83
22d and 23d	9	1.31	2.17	16	0.70	1.43
23d and 24th	9	1.51	2.47	16	1.19	2.18
24th and 25th	9	1.20	1.92	16	0.90	1.53
25th and 26th	10	1.17	1.97	15	0.58	1.38
26th and 27th	12	0.75	1.52	13	0.47	1.13
Totals	67	9.07	15.04	105	5.10	10.01
Means	9.6	1.29	2.15	15	0.73	1.43

Table No. 4 has been prepared to show what water slopes would result if the entrance were contracted by jetties to such an extent that the level of the bay would be practically unaffected by tidal fluctuations in the Gulf. By reference to this table it will be seen that the average ebb slope would be 1.29 feet and would continue 9.6 hours. This is an increase in slope above what at present exists of 1.29 — .62 = .67 feet = 108 per cent., and in duration of 9.6 — 9.4 = .2 = 2 per cent.

The mean of the maximum ebb slopes would be 2.15 feet, which is an increase over what at present exists of 2.15 — 1.21 = .94 feet = 78 per cent.

The average flood slope would be .73 feet and would continue 15 hours. This is an increase in slope over what at present exists of .73 — .43 = .30 feet = 70 per cent., and in duration of 15 — 14 = 1 hour = 7 per cent. It will thus be seen that any contraction of the entrance by jetties will increase the water slope both during the ebb and flood, and the ratio of increase is greater in case of the ebb. This increase of slope continues until the maximum contraction is reached and the bay is practically unaffected by the fluctuation of the Gulf.

It must be remembered that the increased slopes which would result from contraction of the entrance have been compared with the slopes which now exist between the Government wharf and bar. It will be seen that the slope from the gorge to the bar would be increased by a much greater amount.

SUMMARY OF RESULTS RELATING TO GOVERNMENT WHARF AND BAR.

First.—The elevation of the average water level above the mid-plane at the bar is .33 of a foot, and at Government wharf is .23 of a foot.

Second.—The average water level above the average low water is 2.13 feet, and below high water 1.54 feet—a difference of .59 of a foot, equal to 38 per cent. in favor of the ebb.

Third.—The average ebb slope exceeds the flood by 50 per cent.

Fourth.—The average duration of the flood slope exceeds the ebb by 50 per cent.

Fifth.—The mean of the maximum ebb slopes exceeds the mean of the maximum flood slopes by 26 per cent.

Sixth.—The maximum ebb slope exceeds the maximum flood slope by 30½ per cent.

Seventh.—The ebb velocities appear to exceed the flood velocities in a greater ratio than the ebb slopes exceed the flood slopes.

Eighth.—Contraction of the entrance will increase the ebb slopes, and in the case of extreme contraction, to the extent of 108 per cent. The duration would also be increased to the extent of 2 per cent.

Ninth.—The mean of the maximum ebb slopes would be increased 78 per cent. and the maximum ebb slope would be increased 65 per cent.

Tenth.—The average flood slope would be increased 70 per cent. and the duration 7 per cent.

Eleventh.—The mean of the maximum flood slopes would be increased 49 per cent. and the maximum flood slope would be increased 90 per cent.

The conclusion is irresistible, that the building of jetties at Galveston Harbor 7 000 feet apart, with the expectation that the scouring force developed between them would be about as great as it would be were they placed half that distance apart,* must be an error in judgment.

A. E. Kastl, M. Am. Soc. C. E.—It is proper to state here that from the inception of the work by the Brazos River Channel and Dock Company the plane of reference used is the plane of average flood tide of the Gulf of Mexico, which is 1.2 feet above the plane of average low tide.

In the latter part of April, 1891, the writer was appointed engineer to take charge of the completion of the jetties, including the building of the concrete coping walls on those portions mostly exposed to wave action. At the time, it was understood that Mr. Wisner would be the consulting engineer when the concrete coping walls would be started. It was the intention to carry on the work to completion according to the plans set forth by Mr. Wisner, and it was confidently expected that before the next winter's storms set in the jetties would be built up to the water surface, and the worst exposed portions protected by concrete coping walls. In brief, the plans were as follows: Mattresses were to be placed wherever possible and small breaks filled with loose brush—all to be well ballasted with stone; no mattress or brush to be put in place unless an ample supply of stone was on hand to secure the work. Wherever the work reached the water surface it was to be finished with a layer of large stones (one ton and over), and interstices well filled with smaller

* *Vide* report of Chief of Engineers for 1886, Part II, page 1307.

stones. As the old work was sufficiently settled, and as the new work would settle comparatively little, it was the intention to follow up with the concrete coping walls as soon as possible after the crown work was completed. Accordingly work was started on the plans outlined above, and continued until the latter part of May, 1891. The mattresses were built on ways on the river bank or on barges, as most convenient, and then floated into place. Small stone was placed by hand and large stone by means of a steam derrick mounted on a barge. Subsequent events gave strong evidence that these methods of construction were the correct ones.

Some time during the latter part of May, and during the absence of the writer on leave, one of the receivers of the company, and who had been the original contractor,* arrived and assumed charge of the construction. The orders of the writer were countermanded and the plans of the engineers no longer followed. After this, loose brush was thrown in promiscuously and small stone used for ballast. The large stone was unloaded hurriedly and not placed as carefully as it should have been. The writer strongly protested against this method of doing the work, but his advice was disregarded. At this time the writer was transferred to other work and thereafter had nothing more to do with the work on the jetties, which was carried on without the supervision of an engineer.

In July, the writer was informed that a contract had been let for about 2 000 linear feet of concrete coping wall. Subsequently it was given out by the contractors that this would be built on the east jetty from Station 30 to Station 50. The end of the east jetty is at Station 54. In August, 1891, it having become evident that the concrete coping walls, for the construction of which the writer had been engaged, would not be built according to the original plans proposed by Mr. Wisner, and as there were no proper tests being made of the materials to be used, the writer no longer wished to have his name as chief engineer identified with the work, and therefore resigned his position.

About the end of May, 1891, a few weeks after a rise in the river, the least central depth in the channel over the bar was 14.9 feet, opposite and near the end of the east jetty. The distance between 16-foot contours was 275 feet; between 18-foot contours, 500 feet. The mean distance of the outer 20-foot contour from a line drawn across the end of the jetties was 2 320 feet. The area over which measurements were taken extends from a line 300 feet west of and parallel to the west jetty to a similar line 609 feet east of the east jetty. The inner 20-foot contour was 850 feet above the end of the jetties. The channel did not cut out during the rise in the river, but, on the contrary, it shoaled somewhat, and the bar was pushed seaward. The deepening in the channel occurred after the rise and when the velocity of the current was decreas-

* See Mr. Wisner's paper.

ing, but in the meantime the worst breaks in the jetties had been closed, and, consequently, most of the river discharge was forced over the bar. Had the jetties been in as good condition before the rise, the greater river discharge due to the rise would have cut out an 18-foot channel over the bar and carried out the sediment to deeper water; and as at the time there was a strong littoral current to the westward, the sediment would not have been deposited in front of the jetties.

During the first week in July, 1891, there was a heavy northeast storm. When the storm was at its height the tide had risen to a point over 3 feet above average flood tide. Afterwards, a careful examination showed that almost all the loose brush work ballasted with small stones was swept off the crown of the jetties. The writer had always objected to this method of construction, knowing it to be only a temporary expedient and not calculated to withstand heavy wave action. Experience at the Tampico jetties, where the writer had been stationed the previous year, showed conclusively that the crown of a jetty exposed to storms was never safe unless finished off with large stone. Those portions of the Brazos jetties which had been finished off with large stone suffered very little damage.

About the middle of July, after a period of low river, the least central depth in the channel over the bar was 15.5 feet, opposite the end of the east jetty. The distance between 16-foot contours was 150 feet; between 18-foot contours 550 feet. The mean distance of the outer 20-foot contour, from a line drawn across the end of the jetties, determined as before, was 1 780 feet. The inner 20-foot contour was 1 000 feet above the end of the jetties.

A comparison of the results of the two foregoing examinations shows the following: A deepening of the channel over the bar; a decrease in the distance between the 16-foot contours of 125 feet; a slight increase in the distance between the 18-foot contours; a mean recession of the outer 20-foot contour of 540 feet; and a recession of the inner 20-foot contour of 150 feet. The following table shows the recession of the outer contours along the three lines indicated, distances being measured from a line across the end of the jetties.

Contour.	On the Axis of the West Jetty produced.		Recession.	On a line midway between the jetties produced.		Recession.	On the Axis of the East Jetty produced.		Recession.
	May.	July.		May.	July.		May.	July.	
Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
12	1 830	1 630	200	1 370	900	470	End of jetty.		
14	2 010	1 680	330	1 560	1 190	370	End of jetty.		
16	2 230	1 750	480	1 870	1 580	290	100	0	100
18	2 490	1 940	550	2 170	1 830	340	1 150	1 040	110
20	2 730	2 220	510	2 760	2 210	550	2 250	1 750	500

A study of the foregoing table would indicate that the wearing away of the outer face of the bar was evidently due to the action of the westerly gulf currents which prevailed during the time.

The estimate of brush, rock and concrete made in March, 1891, for the completion of the jetties, was as follows :

Brush for both jetties	5 000 cords.
Rock for west jetty, large and small for ballast	4 000 tons.
Rock for west jetty, very large for end	1 000 "
Rock for east jetty, large and small, mostly large	2 500 "
Rock for east jetty, very large for end	1 000 "
Concrete for west jetty	1 000 lineal feet.
Concrete for east jetty	3 000 "

However, it is proper to state that on account of a heavy storm after this estimate was made, the amount of brush and rock would have been increased if re-estimated.

The amount of brush and rock used up to August, 1891, was about as follows :

Brush....................6 500 cords, at $1.50 on barges.

Rock..........9 800 tons, at $3.60, on barges at the works.

No concrete work was done. All the brush was not used in the jetties, part of it being used in wing-dams ; how much the writer is not informed.

About the middle of August, 1891, an examination showed that on the whole the jetties were in better condition than during the previous spring, with the exception of the outer 400 feet of both jetties, which seemed to have deteriorated considerably. Although up to August, 1891, not as much had been accomplished as should have been, still to the work done must be ascribed the improvement in the channel, permitting vessels of 15-foot draught to safely enter the harbor. That better results would have followed if the plans of the engineers had been adhered to and they allowed to control the work, can hardly be questioned. Much work still remains to be done before the jetties can be considered safe from storms. They should be brought up to the water surface as soon as practicable, and wherever exposed to wave action finished off with a layer of large stone (1 ton and over) carefully placed. Then the concrete coping walls should be built on at least the last 3 000 feet of the east jetty and the last 2 000 feet of the west jetty. Until the jetties are so finished there will always be a tendency for the bar to advance with every rise in the river. Then during seasons of low river the action of the sea waves and currents on the outer face of the bar will tend to shoal up the channel at the entrance to the jetties. Whenever the jetties are completed on the plans outlined the next rise in the river will create a 20-foot channel over the bar. The bar material will be

carried out a considerable distance beyond the 20-foot contour, and if the usual conditions prevail, no deposit will take place in front of the jetties. The writer is of the opinion that it will not be necessary to extend the jetties beyond their present length in order to secure and maintain a 20-foot channel. This, however, is conditioned on the thorough completion of the present jetties before the next heavy rise in the river. However, if the works are allowed to deteriorate, a succession of river floods will so advance the bar that it will eventually become necessary to extend the jetties.

The writer is entirely in accord with the principles set forth by Mr. Wisner at the conclusion of his paper, particularly the seventh.

R. E. McMath, M. Am. Soc. C. E.—The history of the improvement at the mouth of the Brazos sets in violent contrast the official and private methods of conducting such an enterprise. A study of the paper shows manifestly that the difference is in the methods and not in the men. The Government engineer is handicapped by restrictions and regulations which leave him little freedom of action, everything must be referred to a distant superior. The whole history enforces the idea that work of this kind should be entrusted to an experienced engineer, who is on the ground taking immediate and responsible charge of the work; one who is observant, quick, and ready in resource, bold to go forward and free to act according to his best judgment.

I therefore heartily concur with the author in the statement made in the seventh and last paragraph in his summary. I can also, from experience, indorse the fifth conclusion, viz., scouring action works up stream.

J. F. Le Baron, M. Am. Soc. C. E.—The abundant success achieved at the Brazos River mouth is peculiarly gratifying to the civil engineers of the country. There is nothing that succeeds like success, and success has been so marked in this case, and the plans so judiciously and wisely designed and ably carried out, that it leaves little room for criticism or discussion. There are, however, a few things that occur to me which it might be well to discuss, and for this purpose one might wish that the chart furnished with the paper was a little more complete and contained more soundings of the adjacent portions of the Gulf, together with the submerged contours and locations of borings, with the line of breakers and some general facts in regard to the force and heights of waves observed there; also of the widths of the mattresses used.

In the sixth conclusion arrived at by Mr. Wisner in his very interesting and instructive paper, he says that the action of the currents and deposits when the east jetty was extended 2 000 feet beyond the west jetty, "very plainly indicate that one jetty would not be very effective for channel making at ports of this class." This assertion may be considered a little too sweeping, and leads us up to the inquiry if a different plan might not have been adopted in this case that would have dispensed

with a portion at least of the west jetty, and so have effected a considerable saving in cost.

It is a well known fact that the concave bends of rivers and training walls always induce a scour and are deeper than other reaches of the river. At Swinemunde, one of the mouths of the Oder River, Prussia, on the Baltic Sea, advantage has been taken of this fact in planning the jetties for the improvement of the channel over the bar. This river is about the same size as the Brazos, with a drainage area of 50 000 square miles, and empties into an almost tideless sea. The prevailing storms come from the northeast, and there is at other times a somewhat strong littoral current to the eastward.

The engineer in planning the east jetty gave it a radius of about 5 500 feet. This is evidently too short, the result being that too much scour has been developed alongside this jetty and deeper water produced than was needed. The east jetty projects about a quarter of a mile beyond the west jetty, which is built nearly parallel to it. The littoral current strikes the concave side of the east jetty in this case, which would be considered unfavorable, but the river current proves strong enough to sweep away all its sediment.

In the case of the Brazos River the littoral current would strike the convex side of the east jetty, provided it had been built on a curve concave to the west. There is another advantage in building such jetties on a curve. The waves rolling in at the mouth will expend themselves on the concave jetty, thus insuring tranquil water in the larger part of the channel instead of rolling up the whole length of the channel to break on the shore. The construction of the windward jetty on a curve entails no increased expense, and if the requisite depth of water is not obtained with one jetty the other can be constructed parallel to it, and in the interests of economy or when quick results are desired to demonstrate the feasibility of obtaining a channel, to a faltering company it seems worthy of trial. Other examples of curved jetties might be cited.

In regard to the width between the jetties, this seems to have been judiciously chosen. It is fortunate that the recommendation of Sir John Coode, of only 350 feet width between the jetties, was not adopted, as in the result of excessive scour and undermining of the jetties, which would undoubtedly have taken place, no room would have been left for building spur walls for protection (as has had to be done even with the adopted width of 600 feet) without fatally contracting the available fairway for vessels. In all such cases I believe in giving a little more width than the result obtained by calculation, as, if found too wide, the channel can readily be contracted by building spurs, which will cause an accumulation of sand, etc., and greatly strengthen the jetty and protect it from the teredo ; but if the fatal mistake is made of building them too narrow, we are left almost at the mercy of the currents.

From the reports of the Chief of Engineers for 1875–79 and 1881, it

appears that the mouth of the Brazos is an exposed place and subject to heavy seas on the bar. The party sent to survey it waited over two weeks for the bar to be smooth enough to allow of venturing out in a boat, and it was then obliged to postpone the survey until a more favorable time. This fact and the fact that the brush mattress jetties built by the army engineers were destroyed show that the waves act powerfully at this place, and that jetties, to withstand their action, must be strong and solidly built. The question arises, if brush and stone jetties will prove strong enough here. In fact it would seem almost the height of imprudence to have used the same class of construction that so signally failed when employed by the Government engineers. After all, it is only a question of stone. If sufficient stone, and large enough, can be piled on the mattresses and kept there, there is no doubt that they will stand; but for several reasons I should prefer in such an exposed position to use log mattresses loaded with stone, such as have been successfully used at Charleston, Cumberland Sound and St. John's River jetties. I was engaged on the last two named works three years from their commencement as principal United States Assistant Engineer.

Both of the places are exposed to the full sweep of the Atlantic Ocean, as the mouth of the Brazos is to that of the Gulf of Mexico. At both Cumberland Sound and the mouth of the St. John's River the object of the improvements is to confine the waters of the Sound and river by parallel jetties where they cross the bar, and so produce a greater depth of water by the induced scour. The bars and headlands at both points are composed of sand. The details of construction have been almost identical in both places. The jetties consist of a foundation course of mattresses from 20 to 87 feet in width, and later up to 120 feet in width, which are sunk on the jetty range and loaded with broken rock to a depth of 12 inches. Successive courses of mats, each 8 feet narrower, are sunk, and each covered with a foot of stone, until the work is brought up to low water. These mattresses were designed by the late General Q. A. Gillmore, M. Am. Soc. C. E. They consist simply of a raft of logs, not less than 9 inches at the small end, which are held together by binders of logs not less than 4 inches at the small end, placed 8 feet apart and spiked to the logs below by 18 or 20-inch spikes. On some of the mats a layer of brush 9 feet wide was placed on the edges, projecting over the sides of the mat, and intended to be 5 inches thick when compressed by the stone. On others the brush covered the whole mat. Poles and sawmill slabs were substituted for the brush in some cases. The brush, especially the 9-foot layer on the sides, was washed off by the waves in a short time. The thickness of the mats was from 16 inches to 22 inches. These mats were very easily made, and required no skilled workmen. No ways were required. The logs were floated down to the log camps, situated 10 to 15 miles up the river, where a shallow shelving beach occurred, and pushed up together, being afloat and held in place by

cant hooks while being spiked to the binders. A well braced mast was set up in the center at each end, the slabs or poles spiked on parallel to the logs, or the brush, when used, bound on with manilla rope, and the mattress was complete.

Seven men would make a mat 100 × 100 feet in three days. The lowest contract price under which these mats were made was 45 cents per square yard. The price varied with different contracts. My estimate of the cost was 35 cents per square yard. Stone was brought out in schooners from New York City and New London, Conn., as ballast, for $1.50 to $1.75 per cubic yard on the vessel's rail. Lime rock from Florida cost $2.45 per ton on the works. It requires some little skill to sink these mats, especially in rough water. This part of the work should be intrusted to good sailor men. The mats are towed out to the works by a powerful steam tug. On each side of the mat is a large, strongly braced decked lighter carrying the stone. Large ship's anchors are previously placed in position, one on each side of the jetty, and buoyed, and the lighters are made fast to these anchors by lines around the capstans, and the mat is hauled up abutting on the last one laid, its position being determined by the masts.

The mat is ranged by its masts, with ranges on shore. During this time the mat is held up by bridle lines at the corners. Stone is now thrown on and the lines slacked evenly until the mat gradually sinks to the bottom. As many as three mats were sometimes sunk at one time. Low water slack is chosen for setting the mats in deep water, and high water slack on the shoals. It is astonishing how accurately mats can be set in calm water by experienced men. But when the tide turns and sets strong before the mat is sunk, it often happened that gaps of 4, 8 or 12 feet would occur, or the mat be as much as this out of line. The plan was therefore adopted of lapping the mats 6 to 8 feet. Sixteen men would set a mat in two and a half to three hours, but in the breakers it took double that time. It was found that the teredo did very little harm to the work. The foundation course was almost invariably sanded over, sometimes as much as 10 feet deep. In the upper courses it was found that the worm ate the exposed ends of the logs and the outer binders, but did not penetrate into that part of the log that was covered with rock. At Cumberland Sound and at St. John's River bar the rock was in a short time completely covered, and all the interstices filled with a luxuriant growth of different species of shell fish—oysters, barnacles, and various bivalves. This effectually protected the logs from the teredo, and induced the deposit of silting sand.

The south jetty at St. John's bar was at one time in imminent danger of being totally wrecked. This jetty crossed the sailing line, as it then existed, and a gap was left for the passage of vessels. The jetty on the shore side of the gap was brought up to low water-line nearly, by about six courses of mattresses, the water being about 22 feet deep. The

result was that the overfall of the current sweeping over the jetty produced eddies and whirlpools that gullied out holes close to the jetty, in some cases over 50 feet deep. The condition of the work then was that of a wall about 20 feet high, standing on top of a sand embankment of about the same width and 9 to 25 feet high. The results were what might have been expected. The currents at once cut through under the jetty, washing away the sand embankment, and sections of the work settled into the gaps thus formed. In one case four courses of the jetty tumbled in, the mattresses tilted until they discharged their layer of stone and the logs floated off, the stone sinking into the live sand and disappearing completely. To arrest this action eight spurs were built, four on each side. These not having the desired effect, an apron 100 feet wide was laid on each side of the work. It consisted of the regular mattresses loaded with stone. This had the desired effect and arrested the scour. The damage was greatly augmented by the works being left without an appropriation for over a year. This system of piecemeal appropriations is most costly and pernicious in works of this kind, and a worse could not easily be devised.

The action just described was observed also at Cumberland Sound, but in a lesser degree, and was checked in the same manner. The jetties at both places were planned with too narrow foundations. Fifty feet was the ordinary width in shoal water, but it was found necessary to increase this to 100 feet, and in the breakers to 120 feet.

The velocities of the currents on the St. John's bar were determined by the U. S. Coast Survey as follows, in feet per second, and are given on its chart:

Ebb, 1st	quarter,	2.03.	Flood, 1st	quarter,	0.84.
" 2d	"	3.33.	" 2d	"	1.66.
" 3d	"	1.15.	" 3d	"	0.84.

The foregoing facts show the necessity of constant watchfulness in building works of this kind, and the impossibility of making an ironclad plan to be undeviatingly followed. They also show the necessity of wide foundations and aprons. The question of log or brush mattressess must be decided for each locality. In some places brush is very plentiful and logs are scarce, but where logs can be obtained at reasonable expense I consider them preferable to brush for mattresses, for the following reasons:

First.—Greater solidity, strength and simplicity.

Second.—Greater facility in making the mattresses, allowing greater speed and less expense.

Third.—No ways are required to make them on, or false works of any kind, and men do not have to be educated in their manufacture.

Fourth.—A jetty composed of logs and stone affords a firmer base than brush, for a stone or concrete superstructure.

The method of making and sinking the mattresses at the Brazos was certainly very ingenious, but the false works and construction of the mattresses under them could hardly be prosecuted in a line of breakers; certainly not in such a sea as we worked in at Cumberland Sound and at St. John's bar. The expense of the false works, track and cars must certainly be considerable ; and it seems, from Mr. Wisner's paper, that there were also required a tug and lighters ; so I cannot see any advantage on the score of economy. Should an extension of the jetties be found necessary, or breaks occur, the trestle and track must be rebuilt the whole distance, or recourse had to tugs and lighters ; and these must be furnished any way for laying apron and spurs. For the reasons stated above I am strongly in favor of sinking mats on ranges, in preference to using a trestle.

From my experience in sea works I am strongly adverse to what is commonly called fine workmanship, or fine joints and carpenter work, or any construction that depends on iron fastenings under water. Only the roughest Titan-like methods will withstand the constant tremors and shocks of the waves. The fine cabinet work executed in making gabions at Galveston is an example. These carefully made cylinders were broken up and destroyed in a single season.

On the Nicaragua Ship Canal (of which I was in local charge in 1888 and 1889), the construction of the breakwater for protecting the channel entrance at San Juan del Norte—the eastern end of the canal—was commenced by building a pile wharf, the bays being filled with brush and broken stone. The outside edges of the wharf were piled close with creosoted piles. The inner piles were uncreosoted. The width of the entire structure was only 40 feet, and the piles were sunk by the water jet, aided by steam pile drivers. The plans of this somewhat novel structure for such a position were sent out from the States. The depth of water was 6 to 18 feet, and it was exposed to the full sweep of the sea. As was to be expected, the teredo speedily found its way between the creosoted piles forming the outer row, and attacked the uncreosoted interior piles. The waves breaking over this frail structure began undermining it, but the shore end was providentially saved by completely sanding over; the foreshore, to the eastward, as had been predicted, advancing rapidly and filling up the windward angle with sand.

In the further extension of this breakwater it is proposed to use stone alone, but rapidity of construction and economy will be secured by using log mattresses as recommended in my design and report on the harbor. The uncreosoted piles for this work cost from $6 to $8 apiece, landed from the ship, before being driven. A log mattress and stone jetty, as originally recommended, would have been far more economical and permanent. A great abundance of black palm logs, impervious to the teredo, besides other suitable timber, exist within half a mile of the work. This work teaches the same lesson : avoidance of carpenter-work

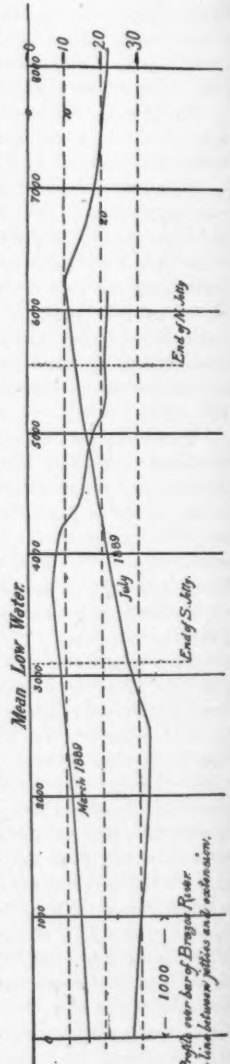
and the necessity of most careful and constant espionage of the work as it progresses, with abundant and ready means at hand for laying aprons for protection from scour.

The experiences at the Brazos River and elsewhere on the Gulf of Mexico emphasize the fact so often stated, that the engineer learns quite as much by the failures of (himself and) others as by their successes; and the result it is hoped will be the adoption of thoroughly rational and successful designs for all improvements hereafter entered upon in these waters or others similarly situated.

W. R. HUTTON,' M. Am. Soc. C. E.—The discordance of the different descriptions of the physical conditions at the mouth of the Brazos River and its bar do not permit an intelligent discussion of the plan of improvement now in progress. The opinions expressed concerning its capabilities for improvement are equally discordant.

In 1872 and 1875, Major Howell, United States Engineers, recommended converging pile jetties by which the depth in the lower river, 15 to 18 feet, would be increased, a channel of somewhat greater depth carried across the bar, sand would be accumulated behind the jetties, and there would be a slow extension of the bar seaward.

Later, Major Mansfield, also of the army, whose recommendations were approved by a board of engineers, recommends parallel jetties of brush and stone; but as the depth of the river mouth is only 10 feet at its most contracted part—this is the greatest depth that can be maintained over the bar. Although there is a slight deposit in bends, there is none near the mouth, and the river carries no sediment. Jetties will transfer the river mouth to the bar, and we must trust to the littoral current to delay its reformation.



Profile over bar of Brazos River
Centre on line between jetties and extension.

In 1887, Major Ernst, United States Engineers, reported the "proposed" improvement as impracticable and any improvement doubtful. That the depth scoured by the

greatest freshets, which could hardly exceed 10 feet, will be reduced to 6 feet or less by the action of the waves, which tend to form a smooth shore line in gentle curves. This they would soon accomplish here if left to themselves. The river alone prevents them.

Mr. Wisner, whose observations are understood to extend over two or three years, sustains Major Howell's opinions, both as to physical facts and as to results.

There seems to be no question that the discharge of the river during a large portion of the year is very considerable, enough to provide amply for any improvement so far as the element of scour is concerned. What is of great importance, but less exactly stated, is the period and duration of low water and its coincidence with other conditions. Mr. Wisner says it covers from one to three months each "season." Does he mean each year? Is it one to three months consecutively? The whole treatment of the case depends on the length of time during which the littoral movements and normal wave action may operate to injure the full river's work.*

The depth curves drawn upon Mr. Wisner's map of 1889 show the existence of a littoral current, as mentioned by him, which causes the deposits to form to the southwestward of the river mouth, the outer face of the bar being quite steep under their influence. The 20-foot curve is not affected by it, for it converges slightly toward the shore to the south, while the other curves diverge markedly from the shore line. The depth lines on the map of 1890, after the construction of the north or east jetty to a length of nearly 6 000 feet, and of the south or west jetty to more than 3 000 feet, show the 20-foot curve to have moved out in front of the jetties some 2 000 feet; the 10 and 15 foot curves show still greater displacement; and a more accentuated divergence from the shore line and a steeper slope of the outer face of the bar.

It is not to be inferred from the foregoing remarks that in a general way the works proposed will not produce the result desired. A further study of the conditions mentioned is nevertheless desirable, that such modifications or additions to the present plans as may be found necessary to insure their permanency, may be introduced in advance and not left until their need is developed by experience.

In all plans for improvement by jetties the character of the material transported is to be considered. At the mouth of the Mississippi the light character of the material is most clearly indicated by the instability of the land formed by it. At the mouth of the Brazos the bar is described as hard sand—though according to Mr. Wisner, the greater portion of the sediment is of a very light character. This is another evidence of the strong coastwise current which carries off the light material.

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* Those two factors, the travel of the sand along the beach and the action of normal waves, are mentioned, but no idea is given of their magnitude or of the conditions under which they operate.

It is to be regretted that both jetties could not have been carried out rapidly and simultaneously to the depth proposed. The late Captain Eads has most strongly emphasized the necessity of extending them at once beyond the bar, so that further sediment brought down by the stream may be discharged in deep water where the transporting currents are strongest.

Mr. Wisner describes the bar as formed by deposits from river floods and by sand drift moved along the coast by current and wave action; he also says that the outer face of the bar has been worn away under the action of waves and currents. It would be interesting to know the conditions under which the same causes produce such opposite results.

GEORGE Y. WISNER, M. Am. Soc. C. E., closing the discussion, writes: Colonel Mendell is mistaken in regard to the alignment of the channel across the Brazos Bar after freshets. A heavy rise in the river always produces a channel curving to the eastward against the littoral current, the same as at the mouths of all rivers flowing directly into an open sea where strong littoral currents exist.

In regard to the low slope of the bottom of the gulf outside of the bar (1 foot in 500), it may be well to remark that it is six times that at Galveston Harbor, which does not seem to produce any fears on the part of the engineer having that work in charge as to the ultimate result.

The question raised by Colonel Mendell as to whether the bar may not reform in front of the jetty entrance is the same as that so strongly adhered to during the construction of the jetties at the South Pass entrance of the Mississippi River. The bar formation to the westward of the South Pass jetties, as has been so well stated by Mr. Ripley, is almost identical in conformation with that at the mouth of the Brazos, and as the currents there for twelve years have produced no indication of the bar reforming in front of the works, there is no reason to fear that any different result will occur at the Brazos. As to whether the littoral currents in the gulf are capable of taking care of the sediment annually carried across the bar by the river, there can be no question, as shown by comparison of the contours of the gulf bottom since 1858. The 20-foot contour is no further seaward in front of the jetty to-day than at that date, and in 1881 was 1 200 feet nearer shore, showing conclusively that the gulf currents are ample to dispose of all material carried by the river. With these facts before us it would seem that Colonel Mendell practically admits that a 20-foot channel is an assured result. Even though the conditions were badly stated in the original paper, it is difficult to infer why he and Colonel Ernst should think that the paper had better have been deferred until final result was obtained; for if such was the case nearly the entire annual report of the Chief of Engineers would be subject to the same criticism, as the harbor improvements there discussed are not likely to be completed for many years.

In regard to the method of mattress construction at harbors on the Pacific coast, reference should have been made to the report of Chief of Engineers for 1890, page 3 022, where the methods now in use there are first described.

It is true, as Mr. Le Baron states, that the concave bends and training walls induce a scour and are deeper than other reaches of the river, but this greater depth is always obtained at the expense of channel width, a very important element at a harbor entrance where strong currents exist. At South Pass the course near the outer end of the jetties has a tendency to make the channel less than the required legal width, and necessitates the keeping of a dredge boat in constant commission to cut away deposits from the convex bend of the channel which invariably occur at certain stages of the river. A curved, narrow channel is very dangerous to enter whenever the current in the river is strong, and such construction should be avoided, if possible.

As to the strength of brush jetties, there is no question that when properly protected with concrete coping walls they will withstand any sea on the gulf coast, but without such protection destruction is inevitable. The cost of construction of one cubic yard of brush mattress, including all false works and superintendence, as may be seen from the table in the original paper, was only 38 cents, an amount that compares very favorably with that of the log mats used by Mr. Le Baron. It is but fair, however, to state that the cost per cubic yard of the completed brush work in the Brazos jetties was only one-sixth of that previously done there by the Government. The work was conducted without difficulty at all times, except during very heavy storms, without loss of material or danger to the crews. Tugs and lighters were used for the reason that all material was brought down the river in that way, and whenever possible was discharged directly into the works from the barges.

The sand drift moving along the shore, which originally was forced out over the bar during freshets, is now arrested by the east jetty and forced up into a high concave sand beach to the eastward of the works. As before stated, this action does not extend much beyond the 18-foot contour, and consequently will play no part in bar formation beyond the jetty entrance. From this I think it will be seen that the same causes are not expected to produce opposite results.

Mr. E. L. Corthell, M. Am. Soc. C. E., sent in a brief discussion, too late to be forwarded to the author of the paper.

He mentions the use of the system of building and depositing the mats adopted at the Brazos as having been in its essential particulars in previous use at Yakima, Oregon, and at other points on the Pacific Coast.

It was adopted at Tampico on works under Mr. Corthell's charge. At Tampico piles were used instead of scantling for grillage, and

"outriggers" were used to give additional width to the mattresses. The last feature was adopted for the purpose of making a much broader base than could be obtained otherwise, except by giving great width to the trestle-work; and it may be stated that by means of these "outriggers" mattresses 80 feet in width, 5 feet in thickness and 60 feet in length have been built at Tampico. These "outriggers," or overhangers, are generally 14 feet in length, built of planks, bolted together; they rest on the end of the caps of the trestles, overlapping the caps to the stringers of the track, and overhanging the ends of the caps, generally, about 9 feet. They are fastened to the caps with chains made for the purpose. There were conditions existing at Tampico which compelled also some different methods than were necessary at the mouth of the Brazos, the former place being much more exposed to the sea; and the work was carried on by means of an ordinary double track railroad trestle, with from six to eight piles in a bent, the center piles generally being creosoted and the brush and stone hauled out over the trestle by an ordinary locomotive and flat cars; although there is considerable work on the south or unexposed jetty, which was done by means of floating equipment, the brush and stone being brought to position alongside the jetty by barges.

Mr. Corthell states that at the South Pass nearly $1\frac{1}{4}$ yards of heavy limestone was used on an average with every cord of brush, although a much less quantity was used in simply sinking the mattresses.

In reference to the conclusions at the end of the paper, Mr. Corthell agrees with them all most heartily. As to the seventh, that the engineer must have full control of all plans and also of the methods, material and rapidity of construction, he agrees with it in general, but says the experience, character and readiness of the contractor to conform to the specifications and instructions given from time to time by the engineer may modify this conclusion.

As to information of the latest date in reference to the channel at the mouth of the Brazos, he is in receipt of a telegram of date of November 27, 1891, to the effect that a vessel of $16\frac{1}{2}$ feet draught has been taken in from sea, with a tide four-tenths of a foot below average flood. When the works shall have been completed and the 20-foot channel obtained, of which there is no doubt whatever, and a great commercial port established, of which, also, there is no doubt in the opinion of Mr. Corthell, a most interesting engineering and commercial paper might, and probably will, be written, of great interest to the Society.

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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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(Vol. XXV.—November, 1891.)

DISCUSSION ON PAPER No. 488—"A MEMOIR ON WATER METERS."

By G. LEVERICH, M. Am. Soc. C. E.

WITH DISCUSSION.

In 1865-66, in levying the United States Internal Revenue tax on whiskey, it was determined that the product should be measured by passing it from each still through a fluid meter carefully enclosed and under lock and seal. To meet the large probable demand for these machines, designs were called for, and in consequence the following was prepared, under date February 26th, 1866, the whole being copied from an old note book, as then recorded.

FLUID METERS.

An automatic measurer of non-elastic fluids, as water, must have a cavity or recess, which, enlarging as the fluid enters and contracting as it passes out, accurately measures the quantity expelled. This variable space may be planned as in two of the accompanying designs, Nos. 2 and 3; consisting of a piston moving backward and forth in a cylinder —appropriate devices opening and closing valves to permit the alternate passing of the fluid and actuated by its weight or pressure; a flexible material may be employed so that the chamber folds up and expands as a bellows; or an elastic fluid, as air, may be used to fill the space left by the water and at a proper time take its place. This last is shown in

design No. 1. Again the movement of the parts may be, as in this design, rotary, and the flow of the liquid be uninterrupted and in one direction, or they may reciprocate either in linear or curved paths, which will cause the flow to be periodic, more or less merged into a continuous one, and the direction will regularly change.

It is desirable in designing a new meter of this class to secure ease and certainty of action, a continuous flow, accurate measurement and simplicity in construction; under all heads it should work well and its cost should be small. Ease and certainty of action are requisite, that light heads may effectually operate the machine, that the wear be slight and that under usual circumstances no stoppage occurs; a continuous flow is desirable, that the pipes be not unduly strained by opening and abruptly closing passages; accurate measurement should be secured by taking care that the parts may not change direction irregularly and that the chambers be uniformly emptied; simplicity of construction is necessary, that the cost of making and of repairing be small and thereby the demand for and use of such machines increase.

With these remarks, or rather suggestions, the designs presented are next to be considered; it being understood that they are crude, and to be modified in proportion and detail as may appear best upon further development and trial of the plans. A careful examination of Patent Office Reports from 1851 to 1862 shows nothing conflicting with their originality upon that record. Of course, in this, as in most similar cases at present, where new designs are made, involving old and much used principles, the originality consists mostly in the arrangement and combination of parts. By way of parenthesis it may be mentioned that Worthington's Patent Water Meter* is the only one in general use.

Design No. 1, as sketched, is shown by Figs. 1 and 2, Fig. 1 being an end view in section through the middle and Fig. 2 a side view, the left half in axial section and the right half a full view. This design consists of a chambered cylinder, made slightly conical and surrounded by a case having an entrance passage on one side and an exit passage on the other, and on the top an air chamber. In the sketch *e* is the cylinder, having eight chambers or cavities (this number may be changed), *m* is the case, *d* the inlet, *f* the outlet, and *a* the air chamber connected with the case by the passages *b* and *c*. The cylinder is closed

* United States Letters Patent No. 13 320, recorded in United States Patent Office Reports for 1855, Vol. I, 718; Vol. II, 184.

at the ends, as shown at one end in the sectional side view, and revolves upon the journal g and pointed pivot i; the journal being packed by the bushing h, which screws into the case and with the pivot i, allows the cylinder to be adjusted endwise, so that it touches the case along the inside and yet easily revolves. It is thought that no packing—that is, of hemp or leather—will be required, and if the workmanship is of good quality, no provision to prevent leakage will be requisite; the friction of movement being small when the cylinder is properly adjusted. This adjustment should be such that a thin film of water or fluid shall separate the rubbing parts, being held there by capillary attraction. Should it be found otherwise, a system of grooves may be cut, as shown in the sketch,

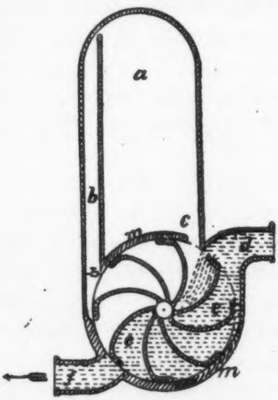


FIG. 1.

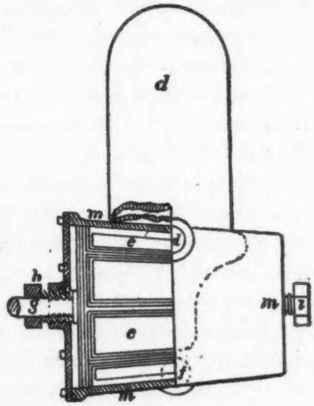


FIG. 2.

which, acting upon the fluid, retard its flow, each similar to a thin plate orifice and furnishing two sharp edges; n being the number of grooves, and f the co-efficient of the head left after passing one, the leakage for n grooves will be expressed by the term $f^n h$. (Similarly this remark upon packing will apply to the other designs wherever grooves are drawn.) It should be stated that this cylinder may be kept adjusted by allowing a slight motion endwise in the journal and pivot, and permitting the fluid to enter the case between its' and the cylinder ends; the difference in size of the two ends causing sufficient difference of pressure to keep the cylinder loosely in contact with the case.

The action of this meter is obvious. To the shaft g any suitable registering device may be attached. A simple one has been planned, but

it is not thought worth while to sketch it here for either of the designs. The capacity and height of the air chamber above the case depend upon the pressure of the fluid to be measured, and whenever it is desirable to preserve the pressure head, the proportions should be such that air in the chamber to be filled by water will, when displaced, compress that in the air chamber so that the water line will not be above the entrance at the case into the passage *c*. For measuring fluids under slight pressure the chamber *a* may communicate with the atmosphere.

The chief doubt in reference to the success of this plan is whether the water may not absorb and carry off some of the air enclosed, so that the water line will be above the orifice of *c*, in which case the passage will cease. A scheme has been thought of, to maintain this line constant under variable pressures, and if the air be absorbed. Of course where slight pressure is had, this doubt does not obtain.

It may be noticed that in the sketch, one revolving chamber is filling and none are emptying. The parts should be so proportioned that the filling and emptying be simultaneous.

Before proceeding to the description of the other sketches, attention may be called to the use of this machine as an automatic boiler feeder. Inlet *d* being closed and outlet *f* connected with the boiler at the water line, steam will displace the water in the chambers (revolved by power) only when water in the boiler is below the orifice of *f*. The passage *b* should connect with a water tank and discharge the steam into it through a syphon top (to prevent water flowing down this pipe), and the passage *c* should connect with the bottom of this tank, it being placed above the boiler. It will be noticed that for each measure of water fed into the boiler an equal measure of steam is allowed to escape into the feed water tank, thereby heating the water, and without attention, the water in the boiler may be kept at a constant level.

Design No. 2, Fig. 3, is given to suggest what should be avoided in practice rather than with any notion of its practical utility. An oscillating meter is shown in side view, the right half in axial section and the left half full, with the cover of the surrounding case removed. This meter consists of a cylinder *e*, containing a loose piston *b*, which together oscillate or tilt on the pivot *g*, to which the register is attached; *d d d'* are the inlet and *f f f'* the outlet pipes, having ports as shown, one discharging or receiving while the other is closed by the wings *c c* of the cylinder; the ports are kept tight by the leather or rubber gaskets

a a, recessed around them and pressed by the fluid tightly against the wings. To prevent tilting before the piston has reached the discharge port, catches are provided to hold the cylinder until released by the piston pressing against them ; these are not drawn.*

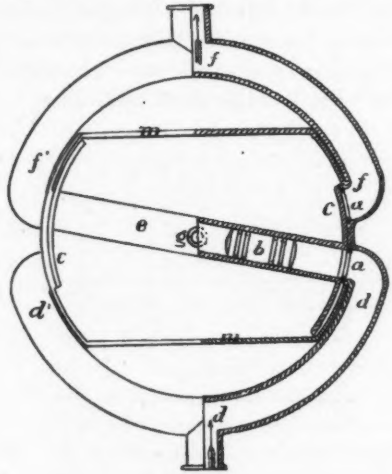


FIG. 3.

Design No. 3, Figs. 4 and 5, is another meter, which is thought well of for water under pressure. Fig. 4 is a side view, the right half in full and the left half with front of case removed. Fig. 5 is a plan of the case, the valves being removed. This meter consists of the case *m*, which is

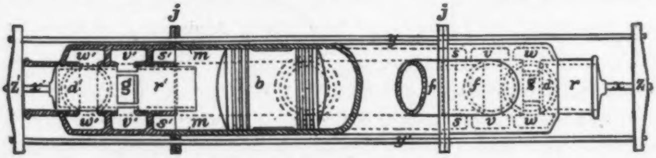


FIG. 4.

a plain cylinder, the inlet pipes, *d d d'* with chambered ports *w* and *w'*, the outlet pipes *f f f'* with chambered ports *v* and *v'*, the loose piston *b* which traverses the cylinder and moves the piston valves *r* and *r'*, said

*Mr. Thompson, the writer of the Paper of which this is a discussion, under date of October 15th, 1891, says : "It may interest you to know, that I only recently had a new meter scheme brought to me, as something which dead sure had solved the problem, that in all essential particulars is identical with that shown in your design No. 2."

valves being hollow and closed at the outer ends, each containing a port s and s' and being connected rigidly together by the rods x x', yokes z z', and rods y and y'; w and w' are inlet ports, v and v' are outlet ports, chambered, or rather connecting with chambers, which are as the ports annular and reach around the piston valves r and r'; s and s' are recesses containing "dead" fluid and will receive and hold sediment until it is removed. The ports in the valves g g' are made somewhat wider than in the case; it will be noticed that the valves are perfectly

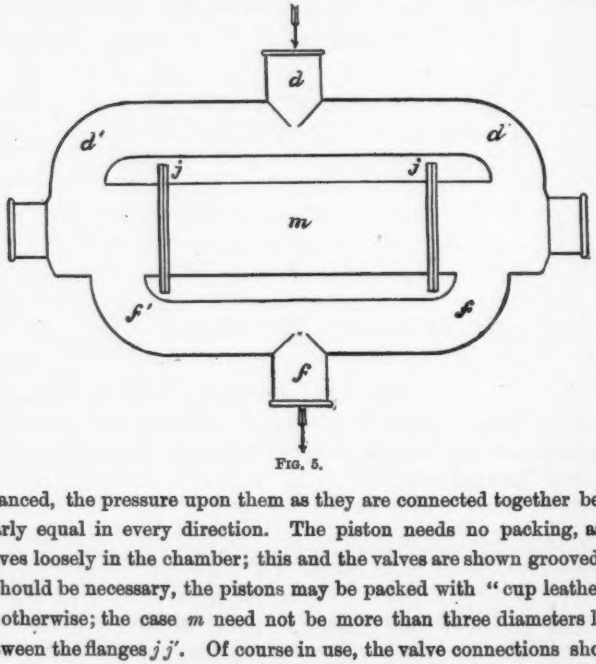


FIG. 5.

balanced, the pressure upon them as they are connected together being nearly equal in every direction. The piston needs no packing, as it moves loosely in the chamber; this and the valves are shown grooved; if it should be necessary, the pistons may be packed with "cup leathers" or otherwise; the case m need not be more than three diameters long between the flanges j j'. Of course in use, the valve connections should be covered to prevent fraudulent tampering with the register, which may be attached to any convenient place on these connections.

The operation is as follows: Fluid entering through the chamber port w, and piston port g, as shown at the right in the sketch, passes toward the left, carrying the piston b before it; the outflowing fluid passing off through the piston valve r', ports g' and v', into the pipe f' and so outward. When the piston b reaches the valve r', which projects into the

cylinder as shown, it carries the valves to the left, closing the port v', nearly closing the port w, opening the port w' slightly, and leaving the port v ready to open. As soon as the port w' is opened, the fluid enters and equalizes its pressure through the port w in a contrary direction, whence the piston b moves a small space further until brought up against the end of the valve casing, thereby closing port w entirely. The fluid pressure now opens wide w' and v and the flow is reversed, the piston b' moves to the right, the operation is repeated, and thus becomes continuous.

Subsequently to writing the foregoing record and previous to December 8th, 1860, two of these designs, No. 1 and No. 3, were elaborated as shown for design No. 1, by Figs. 6 and 7, 8 and 9, and 10 and 11; and for design No. 3 by Figs. 12 and 13, and 14 and 15. These are to scale and were intended as the preliminary drawings of working machines. It is thought that a detailed description is unnecessary.

DISCUSSION.

JOHN THOMSON, M. Am. Soc. C. E.—The author has read with much interest Mr. Leverich's addition to his little paper on water meters; being particularly impressed with the fact that some twenty-six years ago there evidently was as clearly defined a comprehension of the fundamental principles involved as at the present time, when the actual use of and experience with meters is quite out of comparison with that of the date to which this refers.

Personally the author never had experience with the mechanical measurement of the fluid for which Mr. Leverich's designs were specially adapted; although he came rather near to this not long ago, when a well known member of our Society placed an order with his company for a "Cocktail" size; which order, by the way, it was proposed to fill with a meter of 6 inches capacity having an eel trap attached. But seriously, there is a very material difference in the difficulties to be met in measuring fluids of different qualities. Possibly the term "viscosity" would better express the author's present meaning. Thus a meter which may measure water accurately will require special calibrations for oils of different grades, because of the adhesion of the oil to the moving parts, which thus affects the displacement of the measuring mechanism.

In regard to the grooves for preventing leakage past the surfaces of the cylinder, as in Fig. 2, the author has never before seen this feature

more clearly expressed, and believes that the value of this detail in such constructions is not fully appreciated both for the particular function intended by Mr. Leverich and also as a means for relieving such actions when obstructed by the introduction of fine particles of sand. In a recent experiment upon this very detail, using a freely fitted plug, first tried smooth and then grooved, with a difference of pressure equal to about 20 pounds to the square inch, the leakage past the plug when grooved was, as is recollected at this writing, nearly 20 per cent. less than that past the plug when smooth without the grooves.

The design illustrated in Figs. 1 and 2 is entirely new to the author, and he has no recollection of having seen a similar scheme. It is novel and interesting, although in the author's opinion it would not have been a satisfactory device in practice, for the reason that the wings of the rotating cylinder would tend to exhaust the air from the reservoir *a*, even if the air were not absorbed, which it probably would be, and also that the cylinder is seriously out of balance, the thrust being from the inlet pipe *d* to the outlet *ff*.

The meter shown in Figs. 4 and 5 is illustrative of a type upon which much labor and thousands of dollars have been unsuccessfully expended. To render it certain of operation at all rates of flow it would require the addition of the duplex piston and valve of Worthington or the gravitating weight of Kennedy. Without such auxiliary means the reversal of the valves depends upon the momentum of the piston; wherefore if the friction of the piston is equal to or greater than its momentum, the valves, if made to lap the ports, will close off and stop the flow; or if constructed without lap, will permit the fluid to pass free.

But it is comparatively an easy task to criticise an early work through the "hindsight" of after experience. It has been aptly said that "To rightly propose a problem is a considerable step toward its solution;" and such a proposition to the end in view Mr. Leverich at least came very near to making, if he does not in fact submit it in the opening paragraphs of his present discussion.

ERRATA.—Vol. XXV, p. 447.—15th line from top 3 400 yards should read 400 yards.

A late letter from Mr. Edwards says: "The capacity of the dredging steamer 'Advance' is 500 yards, and she loads herself in Gedney's Channel (one of the entrances to New York Harbor) on an average in 40 minutes. The capacity of the 'Reliance' is 650 yards and she loads herself in 45 minutes."

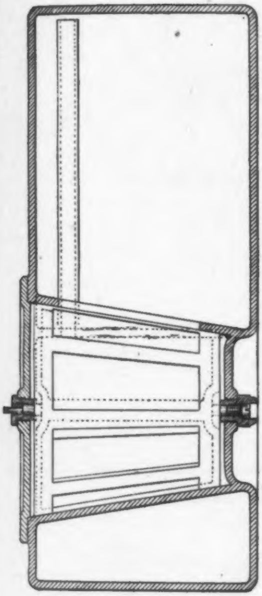


FIG. 6.

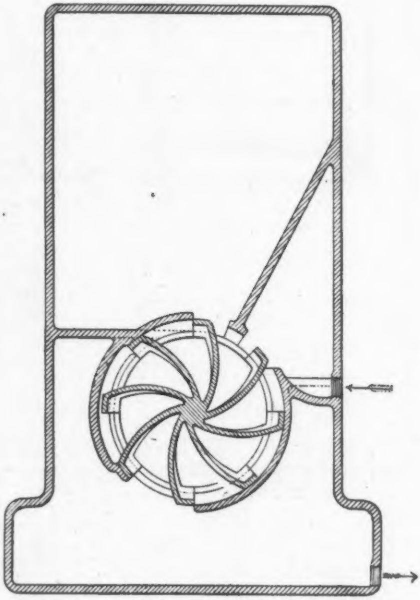


FIG. 7.

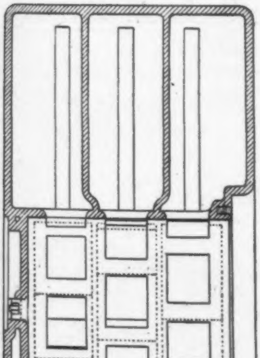


FIG. 8.

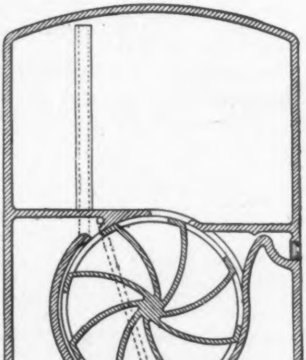


FIG. 9.

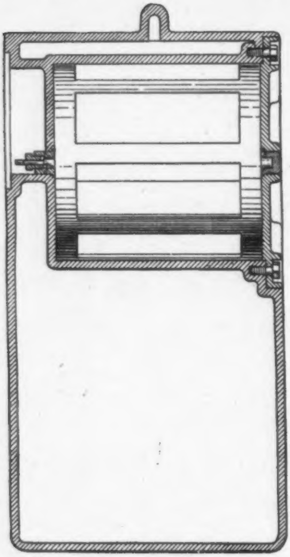


FIG. 10.

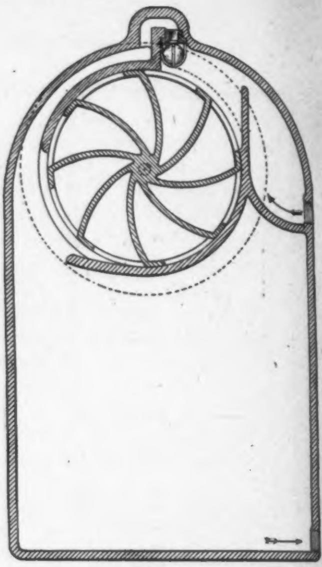


FIG. 11.

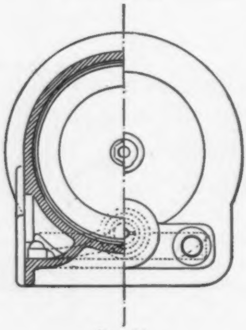


FIG. 12.

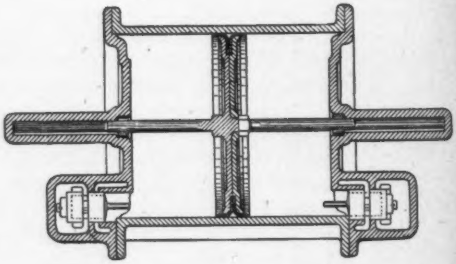


FIG. 13.

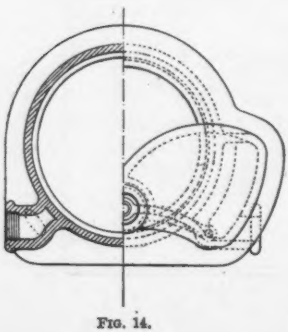


FIG. 14.

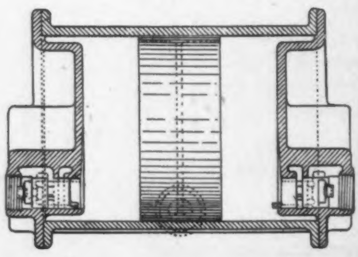


FIG. 15.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

513.
(Vol. XXV.—December, 1891.)

THE IMPROVEMENT OF THE CHANNELS AT THE ENTRANCE TO THE HARBOR OF NEW YORK.

By JOSEPH EDWARDS.

When it is considered that the City of New York, with its adjacent cities, is soon to become, if it is not already, the greatest commercial center of the world, and that its social as well as business relations with all nations is rapidly increasing, it is evident that nothing can be of more importance to the interest, not only of the City of New York, but of the whole country, than an ample waterway between its docks and the ocean, of such depth and width that the largest ships of these and future times may pass into and out of its harbor, at all states of the tide, as safely and freely as ferry-boats are run on its surrounding bays and rivers.

Introductory to this paper it seems desirable to call attention to the commanding importance of the improvement:

"At this port two-thirds of the merchandise imported into the United States are received, and two-thirds of the import duties are collected. From this port are sent out one-half of the domestic products of the country which are exported, and here one-half of the foreign tonnage trading with the United States enters. Three-quarters of the passengers traveling between the United States and foreign countries come and go

by way of New York, and three-fifths of all emigration land at Castle Garden."

The following are the statistics of the commerce of the harbor for the fiscal year ending June 30, 1885:

Total receipts from all sources	$126 183 873 57
Value of imports	380 075 748 00
Value of imports for interior ports	16 833 562 00
Value of imports in transit	16 052 183 00
Value of domestic exports	334 729 775 00
Value of foreign exports	9 796 534 00
Value of merchandise in bond June 30th	21 485 083 00
Value of specie (imports)	6 314 264 00
Value of specie (exports)	14 656 718 00

	Tonnage.
Foreign vessels entered, 4 130	4 729 283
Foreign vessels cleared, 4 031	4 670 360
American vessels from foreign ports, 1 691	930 444
American vessels for foreign ports, 1 273	770 105
Coastwise vessels entered, 1 913	1 761 348
Coastwise vessels cleared, 3 076	2 280 063

Channels of New York Harbor.—The several channels of New York Harbor below the Narrows, and the work performed on them, are clearly shown on the accompanying map, which was prepared from a special survey made by Maj. G. L. Gillespie, Corps of U. S. Engineers, in 1884, by direction of the Government, with reference to this improvement, the figures showing the soundings having been changed to correspond with the depth and width of the channels as now improved, and the location of the work designated by checked lines.

Col. Gillespie's report of 1890 states as follows:

"The lower bay is a large tidal basin, with an area of about 100 square miles." The distance by the main ship channel, "from the Battery at New York to 30-foot soundings outside of the bar of Gedney's Channel, is 22 miles, and by the Swash Channel it is 18 miles, or from the Narrows 15 and 11 miles respectively. From the northeast around by the east to the southeast the lower bay is open to the full sweep of the Atlantic Ocean. From the Narrows to the northern point of Sandy Hook is about 9 miles, but the shortest distance across the bay, from the point of the Hook to Coney Island, is 7 miles. Below the Narrows there is one main channel, known as the main ship channel, running

southward to a point about 1 mile west of the upper end of Sandy Hook; thence turning northward and eastward for 4 miles to the head of Gedney's Channel, and thence through Gedney's Channel east to deep water of the ocean. Gedney's Channel is the main channel across the Ocean Bar, lying at the entrance to New York Harbor, about 3 miles outside of Sandy Hook, and east by north from it. The Swash Channel is really a cut-off from the main ship channel, leaving it about 6 miles below the Narrows, and joining it again at the western end of Gedney's Channel."

Lying northward of the above described channels, and extending eastward, are three other lesser channels, the "Coney Island" (which is used only by local excursion boats and small sailing vessels), the "Fourteen-Foot" and the "East Channels," for location and relative position of which see map (Plate CIII).

Relating to the depths of water in the upper bay, the report states:

"From the Narrows northward to New York City there is no water less than 6 fathoms (36 feet) in depth in the main channel."

Condition of the Channels before Improvement.—Colonel G. L. Gillespie, in his report for 1890, says:

"Before the improvement of the Main Ship Channel into New York Harbor was undertaken by the United States, it was obstructed by four shoals as follows:

"*First.*—The outer bar, about 4 000 feet wide, the channel across which is known as Gedney's Channel, where there were depths 23.7 feet in mid-channel and 22.3 feet in the southern half.

"*Second.*—The shoal at the mouth of the Swash Channel, about 4 000 feet wide, where the depth was 24.3 feet.

"The channel across this shoal has been named the Bayside Channel.

"*Third.*—The shoal northwest of Sandy Hook, about 2 000 feet wide, on which the least depth was 26.2 feet.

"*Fourth.*—The shoal in the Main Ship Channel in the lower bay, west of Flynn's Knoll, nearly 3 miles long, on the crest of which the depth was only 23.9 feet in mid-channel, with depth of 22.6 feet within a few hundred feet of the mid-channel range.

"A large proportion of the vast commerce of the port, which is carried on in vessels of great draught, could only cross these channels at, or near, high water."

Referring to the condition of the channels before the improvement was made, Lieutenant-Colonel McFarland, in his report of 1886, says:

"Major Gillespie, in 1884, reported that the deepest draught vessel that had ever entered New York Harbor was the Spanish frigate *Numancia*, which drew 28 feet 8 inches, but that she had to wait for a

spring tide to cross the bar. The minimum depth of water in the Main Ship Channel at mean low water in February, 1886, was 23.3 feet, to which must be added 4.8 feet for mean rise of tide, which would give a minimum depth of 28.1 feet at mean high water, or only one-tenth of a foot more than the draught of the largest ocean steamers at present. Slack water at Sandy Hook averages about forty minutes, so that the vessels of deep draught have only forty minutes in which to cross the bar in Gedney's Channel and to pass the Knolls, 7 miles above it. It is common to see vessels passing over the bar with a wake 500 to 1 000 feet long behind them of material churned up from the bottom by their propeller blades. Thirty feet at mean low water appears to be the depth which is required to enable the largest ocean steamers to come in with 2 feet to spare below their bottom."

First Action Taken by the Government.—Though the need of deeper channels in New York Harbor had been yearly and rapidly increasing, and had been frequently agitated by the commercial bodies of New York, the Government, though from time to time solicited to provide increased facilities, took no action, and up to July 5th, 1884, no money had been appropriated for the improvement of the lower bay and its channels, and no work had been done. But on that date, without any preliminary examination, survey or report, an appropriation of $200,000 was made by Congress for deepening Gedney's Channel, New York Harbor. At this time the engineer in charge, Major G. L. Gillespie, "was authorized and directed to make a survey of the lower harbor from the eastern end of Coney Islannd to Sandy Hook, with a view of determining the most feasible plan of improvement. The survey was begun in August and completed in November, 1884, and transmitted to the Chief of Engineers, December 6th, 1884."

Projects Proposed by Government Engineers.—When the improvement of the harbor thus came to be practically considered by the Government, the problem to be solved by its engineers related to the method to be adopted, which, as far as could be determined without experiment, would best attain the desired results. This involved several considerations bearing upon whatever plan might be projected:

First.—The probability of successfully obtaining the required depth of water.

Second.—The probability of permanency of deeper water after the completion of the work.

Third.—The relative cost of the different plans.

First Project.—Major Gillespie accompanied the report of his survey, above mentioned, with various observations relating to what he found

to be the condition of the channels as compared with their condition at previous times; the velocity of the currents, the character of material found in the different channels, and various other important information relating to the proposed improvement; together with his project for improving the channels and the estimated cost, as will appear from the following extracts from his report for 1885:

"The act of July 5th, 1884, makes a specific appropriation for the deepening of Gedney's Channel, and this project, therefore, is limited to that channel. The methods usually adopted for deepening a channel are by contraction or by removal of material by steam dredges."

Referring to contraction by constructing a stone dike to spring from Coney Island and extend several miles toward Sandy Hook, and strengthening Sandy Hook to prevent it being washed away, Major Gillespie says:

"Before considering such a project it would be well to try the experiment of deepening the channels across the bar by dredging, which has been so successful, by report, at the mouth of the Tyne, and at other ports in England, where the dredged channels have been in the open sea, and when improved, have been self-maintaining. If the experiment fails here, then the composition and nature of all the shoals should be accurately determined by deep borings, the action of the currents definitely learned, and the method of contraction studied.

"The project which I submit for present consideration, then, is to open a channel through the shingle shoal lying across the western entrance to Gedney's Channel, between the 30-foot curves, low stage, by dredging, or by any other method provided for the raising of the obstructing material and carrying it elsewhere to an assigned place of deposit, or by any well developed plan of removal by artificial currents. The proposed cut will extend along the axis of the channel for an approximate distance of 4 000 feet, will be 1 000 feet wide, and will carry 30 feet at mean low stage. The maximum depth of cutting will be 6.5 feet, and the amount of material required to be removed will be 700 000 cubic yards measured in place. I do not know that the cut, once opened, will be self-maintaining, but the present appropriation being small, it is well enough to experiment with it, and if the experiment is moderately successful, the use of large sums for contraction by stone structures may be avoided, and the annual appropriation for maintenance may be placed at comparatively low figures.

"There is a shoal in the main channel west of Flynn's Knoll, 2 miles (approximately) wide, mean low water, with only 25 to 25.5 feet deep. To deepen the channel there to 30 feet mean low water will require an improvement similar to that in Gedney's Channel; but as it is inside of the bar, and there is a safe and deep anchorage for vessels near the Hook,

the improvement is not so urgent as in the latter channel; still, there are other considerations which make it highly important that this shoal should be removed simultaneously with the improvement on the bar."

COST OF THE IMPROVEMENT.

Gedney's Channel:	
Channel 1 000 feet wide, 30 feet mean low water, 700 000 cubic yards, at 50 cents	$350 000
Channel 1 000 feet wide, 28 feet mean low water, 385 000 cubic yards, at 50 cents	192 500
Channel 1 500 feet wide, 27 feet mean low water, 367 000 cubic yards, at 50 cents	183 500
Channel 1 000 feet wide, 27 feet mean low water, 200 000 cubic yards, at 50 cents	100 000
Channel 800 feet wide, 27 feet mean low water, 146 000 cubic yards, at 50 cents	73 000
Main Channel, West of Flynn's Knoll:	
Channel 1 000 feet wide, 30 feet mean low water, 1 550 000 cubic yards, at 40 cents	$620 000
Channel 1 000 feet wide, 28 feet mean low water, 794 000 cubic yards, at 40 cents	317 600
Channel 1 500 feet wide, 27 feet mean low water, 738 000 cubic yards, at 40 cents	295 200
Channel 1 000 feet wide, 27 feet mean low water, 467 000 cubic yards, at 40 cents	186 800
Channel 800 feet wide, 27 feet mean low water, 373 000 cubic yards, at 40 cents	149 200

RECAPITULATION.

Improving Gedney's Channel for 30 feet, mean low water	$350 000
Improving main channel for 30 feet, mean low water	620 000
Total	$970 000

"No allowance has been made in the above computations for irregularities in cutting, or for increase in bulk should the material be measured in scows. This increase would amount probably to 30 per cent. for the 28 and 30-foot depths, and to about 50 per cent. for the 27-foot depths.

"The least width for a conveniently navigable channel, where cross currents exist, for the largest class of vessels, is 800 to 1 000 feet. I

recommend that the available funds be applied toward the opening of the 30-foot channel, by dredging, for a width dependent upon the cost of removal of material, and the work be done by contract, after soliciting sealed proposals by public advertisement in the usual way. I also recommend that Congress be asked to make an additional appropriation of $770 000 this session, to complete the improvement proposed both for Gedney's Channel and for the main channel on the inside, or for application toward the commencement of permanent works of construction, if such works be found necessary. It is my belief that if before any work is done a sufficient time be given to the contractors to prepare an extensive plant, suitable for making the improvement rapidly, the money appropriated will be most judiciously and advantageously expended, and that it may reasonably be expected that the channel will, after the proposed improvement has been effected, be self-sustaining for many years."

It seems reasonable to suppose that the above described project, suggested and recommended by Major Gillespie, determined the method of improving New York Harbor; and though it was at first only experimentally adopted, yet by its final and complete execution it has resulted in preventing an unnecessary outlay of a large sum for contraction works, and avoiding any detriment to the Lower Bay that might have resulted therefrom, as well as consummating the desired results.

Second Project.—Major Gillespie's project and recommendation was submitted, by the Chief of Engineers, to the Board of Engineers, December 10th, 1884. The Board submitted its report thereon December 23d, 1884, which is too extended to be quoted here in full. After concluding that the desired improvement could be best obtained by contraction, it says:

"Dredging would be needed through the 25-foot shoal west of Flynn's Knoll. Colonel Gillespie's estimate for a channel through this shoal, 1 000 feet wide and 30 feet deep, is $620 000. If after the construction of the dike the main interior channels should remain as they now are, this dredging would form a part of the plan; but as that is uncertain, and as much relief would be afforded here by a clear channel 1 000 feet wide and 28 feet deep, in advance of the completion of the 30-foot channel on the bar, dredging to that extent is recommended. Colonel Gillespie estimates its cost at about $420 000. The total cost of the improvement, giving 30 feet deep from New York to the ocean, would be about $5 000 000 or $6 000 000. The Board of Engineers has little expectation that anything more than temporary relief can be obtained by dredging on a bar exposed to the full force of the Atlantic, and hence cannot recommend that method for a permanent improvement. As already stated, it should be used at Flynn's Knoll.

"Having considered the problem of improving the entrance to New York Harbor in a general way, attention will now be turned to Colonel Gillespie's project for expending the $200 000 appropriated by act of Congress approved July 5th, 1884, for Gedney's Channel. While not expecting large results from dredging here, yet, as the appropriation is specifically confined to this channel, the Board recommends that an attempt be made to secure a channel here 28 feet deep, and of such width as the appropriation will pay for, by one of the numerous methods of dredging. If before the whole appropriation were expended experience should show that the dredged channel rapidly filled up, the work might be stopped by the Secretary of War, the contractor being properly remunerated.

"To recapitulate: The Board recommends as a general plan for improving the entrance to New York Harbor so as to give 30 feet from New York to the ocean, the construction of a stone dike running about south south-east from Coney Island to such distance as shall be found necessary, and probably not less than 4 miles; the protection of the head of Sandy Hook; and the dredging of a 30-foot channel from the deep water near Sandy Hook to deep water below the Narrows; also the immediate dredging of a channel 1 000 feet wide and 28 feet deep through the shoal west of Flynn's Knoll, as soon as Congress shall furnish the funds; also that the existing appropriation be applied to dredging Gedney's Channel to a depth of 28 feet."

Increased Estimate.—Referring to Major Gillespie's estimates, made and transmitted to the Chief of Engineers December 6th, 1884, amounting to $970 000, as the probable cost of dredging the two channels, Colonel McFarland, in his report for 1886, states that Major Gillespie's estimates were too small, and says "no allowance is made in them for irregularities of cutting or for increase of bulk, should the material be measured in scows, and that this increase would amount to 30 per cent. for a 30-foot channel. Actual experience in dredging from February, 1886, to June 24th, 1886, shows this increase to be roughly as 55 to 40" (equal to 37½ per cent. more), "so that to Major Gillespie's estimate should be added, in round numbers, $150 000 for Gedney's Channel, and $250 000 for the Knolls, making the totals:

Gedney's Channel...........................	$500 000
The main channel..........................	870 000
Total..................................	$1 370 000

"This is the amount required at Major Gillespie's estimate of fifty cents per cubic yard in Gedney's Channel and forty cents per cubic yard in the main channel."

Why Dredging only was Finally Adopted.—As the appropriation of the $200 000 granted in 1884 was made for dredging in Gedney's Channel, this money could not be applied to the general improvement in any other way; therefore, dredging in this channel was commenced before the project for contraction was either adopted or abandoned. And before the question of contraction, at the estimated cost of $4 500 000, was wholly dismissed, the work that in the mean time had been done and was being done on this channel foreshadowed the possibility of accomplishing the entire contemplated improvement by means of dredging alone. The Board of Engineers, therefore, recommended that the dredging be further extended, and that if the results continued to be favorable and altogether successful, the delay and cost of constructing contraction works might be avoided.

At the beginning of November, 1886, 303 869 cubic yards had been dredged from this channel, and some doubt existing as to whether the shifting of sand during winter storms would not again fill up the cut, arrangements were made for determining this question by having comparative surveys made. One of these was taken between the 17th of October and 2d of November, 1886, and the other in May and June of 1887, by Lieutenant Derby, of the Corps of Engineers, who had been placed in local charge of the work.

The permanency of the results of dredging, as shown by these comparative surveys, will be seen by the following quotations relating thereto, from Colonel McFarland's report of 1887:

"The depths were found to agree almost exactly with the depths given by the survey made in the fall, the only difference being that those obtained in the spring were found to be one or two-tenths of a foot greater. This is a very satisfactory result, for it shows conclusively that for eight months, including the stormiest season of the year, the channel has maintained the increased depth which it had received; and it leads to the belief that the still greater depth which the act of Congress calls for may be equally maintained when once secured."

Besides the above and other evidence that Gedney's Channel could be successfully and permanently improved by dredging alone, as first projected by Colonel Gillespie, the plan by contraction had met with opposition from the New York Chamber of Commerce. There was apprehension that the contemplated contraction works, in the form of a half tide dike springing from Coney Island and extending several miles toward Sandy Hook, would obstruct navigation through other channels

largely employed by coastwise vessels, and that it might otherwise result in damaging the harbor. The communications between the Chamber of Commerce and the Board of Engineers are too lengthy to be here further alluded to. The report of the Board of Engineers, made December 11th, 1886, though somewhat conservative on the project embracing contraction, was as follows:

"The project presented by the Board of Engineers in its report for December 23d, 1884, for securing a permanent low water channel 30 feet deep from the deep water of New York Harbor to the deep water of the Atlantic, by way of Sandy Hook, is the most certain method of accomplishing the end desired; and, as it provided for the expenditure of more than $1 000 000 in dredging, it is recommended that the existing appropriation of $750 000, which was made under the project, and in accordance with the recommendation of the Chief of Engineers, transmitted to Congress by the Secretary of War, be applied wholly to dredging in Gedney's and the main ship channels. It should be stated that the dredging was begun in Gedney's Channel because Congress restricted the use of the appropriation of $200 000 first made to that entrance. Its deepening by dredging having thus been begun by the direction of Congress, should now be thoroughly tried. Should experience show, and only experience can show, that the cost of maintaining a dredged channel through the outer bar will not be excessive in comparison with that of contraction works, then dredging will be by far the best means of permanently improving the entrance, and it would give the speediest relief to navigation. But if the cost of maintaining the channel prove excessive, then the remainder of the plan proposed by the Board of Engineers in 1884 will come up for execution."

Progress and Testing of the Work.—As the work progressed from year to year, various careful comparative surveys were made to ascertain whether the depths of water attained were being maintained, and if not, to what extent they had been diminished. These surveys invariably proved, that while the depth of water had in no part of the channels become less, it had in some places become a trifle greater. The report of the Engineers for 1888 says:

"Surveys made in December, 1887, and May, 1888, show that no shoaling whatever had taken place on the bar in the interval of six months during which no dredging was done there. As a like comparison was made a year ago, with precisely the same result, there are good grounds for expecting that the dredged channel across the bar may maintain its new dimensions by the action of the current alone."

The report for 1889 says:

"A survey of the Main Ship Channel from below the Narrows out

along the improved channel to deep water beyond the bar, was made in June, 1889, and the resulting charts will soon be published for the information of mariners. These charts, which were first made in December, 1888, in separate sheets, covering the several sections of the improved channel, have been eagerly sought after by all the steamship companies of the port, to whom they have been liberally distributed free of charge. The survey just completed shows that the improvement is in the most gratifying condition. There is no indication that Gedney's Channel has shoaled since the last survey of December 29th, 1888, when the least depth in the channel width of 500 feet was 30 feet at mean low water. The Bayside Channel is entirely free from the small shoal spots which formerly existed in it, at or near the eastern entrance to the Swash Channel, and the line of deep water is now direct from the western entrance to Gedney's Channel, westward to the southern entrance to Main Ship Channel opposite to red Buoy No. 10, and the least depth throughout the entire width of 1 000 feet is 30 feet at mean low water.

"The Main Ship Channel, west of Flynn's Knoll, from buoy No. 10, the northern limit of the 30-foot curve in Sandy Hook Bay, to Buoy No. 12, the extreme northern limit of the present improvement, has 29 feet at mean low water between parallel lines 50 feet and 500 feet, respectively, west of the line of buoys *C*2 and *C*6. The 30-foot channel between the same extreme north and south points has an average width of 350 feet. When it is remembered that before this improvement began, in 1885, the least depth in Gedney's Channel was 22.3 feet, in Bayside Channel 24.3 feet, and in the Main Ship Channel, west of Flynn's Knoll, 22.6 feet—all at mean low water—the great results attained by the work just reported will be quite apparent. The noticeable result is that there is now (1889) a navigable channel from the wharves at New York City to the sea, affording 30 feet depth, approximately, at mean low water, and 34.8 feet at high water, and that it is practicable for the largest steamer which visits the port to pass in or out over the bar in fair weather without regard to the tides."

Relating to the durability of the improvement, Colonel Gillespie, still later on, in his report of 1890, says :

"Surveys of all the channels undergoing improvement were made in July, 1889, and again during January and February, 1890. These surveys show that the improvement is in a very satisfactory condition. Gedney's Channel and Bayside Channel (east and west) are practically completed, having a depth of 30 feet at mean low water, for the full projected width of 1 000 feet. The Main Ship Channel west of Flynn's Knoll has a depth of 30 feet, mean low water, for a width of 500 to 800 feet, and a depth of 28 feet for a width of 800 feet throughout. The severe storm of September 9th, 1890, which caused a suspension of work

for one week, does not appear to have had any effect on the improved channels. There is no evidence of shoaling, and the soundings of the various surveys agree so well with one another that it seems highly probable that the improved depths will be well maintained."

At the date of writing this paper there is a continuous channel 30 feet deep at low tide, and 1 000 feet wide from the Narrows to the ocean, and the largest steamships can enter and leave the port at any hour of the day and all states of the tide.

The several appropriations by Government for improvement were as follows :

For Gedney's Channel : by act of July 5th, 1884......	$200 000
For New York Harbor : by act of August 5th, 1886....	750 000
For New York Harbor : by act of August 11th, 1888...	380 000
For New York Harbor : by act of September 19th, 1890	160 000
Total....................................	$1 490 000

Cost of the Improvement.

Estimate for Contraction Jetties.—The project, by means of contraction works, for the improvement of the outer bar, would have involved the construction of a stone dike extending some 4 miles across the shoals of the bay from Coney Island toward Sandy Hook, with a suitable stone protection for the head of Sandy Hook to prevent its being scoured away by the increased current, at an estimated cost of $4 500 000. Besides the requirement of this dike, it was estimated that dredging through the shoals of the Main Ship Channel, which it was supposed the dike would not improve, would have made the cost of the entire improvement, approximately, $6 000 000. (See table.)

Estimate of Cost by Dredging.—As estimated by the Government Engineers, Colonels McFarland and Gillespie, the whole amount of dredging required to complete a continuous channel from the deep water of the ocean to the Narrows, not less than 1 000 feet wide and not less than 30 feet deep at mean low water, would be 4 300 000 cubic yards ; which, at the price (fifty-four cents per cubic yard) paid on the first contract given out on this improvement, would amount to $2 322 000. Respecting the cost of such a channel by dredging alone, Colonel Gillespie, in his report for 1885, estimated fifty cents per cubic yard for Gedney's Channel work and forty cents per cubic yard for that of the Main Ship

Channel. But in 1886 the engineers estimated the cost between thirty-four and thirty-five cents per cubic yard, and that, at this price, the improvement would cost $1 490 000.

Colonel Gillespie, in his report for 1890, says :

"The estimated cost for opening the channel by dredging, revised in 1886, was fixed at $1 370 000, which was again increased in 1887 to $1 490 000."

After the improvement began it was found essential to extend the Main Ship Channel north of Buoy No. 12, to remove the Northwest Shoal, to deepen the Bayside Channel and to extend the Gedney Channel, none of which were included in the original project, and which account for the subsequently increased estimates.

Actual Cost.—The improvement having been made under several different contracts, varying in number of yards and in price per yard, the entire cost is, of course, made up by the aggregate amount paid on all the several contracts, which is $1 285 682 94 for the removal of 4 875 079 cubic yards (instead of 4 300 000, as first estimated it would be necessary to remove), being an average of 26.4 cents per cubic yard, and only 23.37 per cent. of the originally estimated cost.

The tabulated cost is as follows :

	No. of cubic yards removed.	Average price per cubic yard.	Total cost.
Main Ship Channel	3 201 411	25.26 cents.	$808 850 71
Gedney's Channel	1 673 668	28.5 cents.	477 012 23
Totals	4 875 079	26.4 cents.	$1 285 862 94

Economy of Dredging.—By executing the entire work on both channels exclusively by dredging, instead of contraction supplemented by dredging, not only has the Government saved an unnecessary expenditure of $4 594 701 46, as shown by the following tabular statement, but much less time has been required in satisfactorily completing the improvement, and thus sooner providing the much needed better facilities of navigation, as well as avoiding the apprehended detriment to any of the several channels of the harbor.

Besides this large amount of saving by means of dredging, the Government, by a fortuitous circumstance, obtained some advantage by

dredging in portions of the Main Ship Channel from cross currents, as explained by Colonel McFarland in his report for 1888.

By the same tabular statement will be seen the difference between the various estimates of the cost by dredging only, and the actual cost.

The first official estimate (Report for 1885) for dredging Gedney's Channel was fixed at fifty cents per cubic yard, and for the Main Ship Channel at forty cents; while the average actual cost on the Gedney's Channel has been but 28.5 cents per cubic yard, and on the Main Ship Channel 25.26 cents, which aggregates a difference of $831 535 46.

This table also shows that, even at the revised estimate, subsequently fixed at 34.5 cents per cubic yard, including both channels, the improvement has been completed at a cost of $396 039 31 less than the lowest official estimate.

Tabulated Statement, showing the difference between the estimated cost by the several methods proposed and the actual cost by dredging, based on the number of yards (4 875 079) actually removed.

Project Recommended and Location of Work.	Estimated Prices for Dredging.	Estimated Cost.	Actual Cost—all being done by dredging.	Difference—Saving to the Government.
Contraction work for improving Gedney's Channel	....	$4 500 000 00	$477 012 23	$4 022 987 77
Dredging in Main Ship Channel, 3 201 411 yards	At 40 cents..	1 280 564 40	808 850 71	471 713 69
Totals	....	$5 780 564 40	$1 285 862 94	$4 494 701 46
Dredging in Gedney's Channel, 1 673 668 yards	At 50 cents..	$836 834 00	$477 012 23	$359 821 77
Dredging in Main Ship Channel, 3 201 411 yards	At 40 cents..	$1 280 564 40	808 850 71	471 713 69
Totals	....	$2 117 398 40	$1 285 862 94	$831 535 46
Dredging in both channels, 4 875-079 yards	At 34.5 cents	$1 681 902 25	$1 285 862 94	$396 039 31

Referring to the material in Gedney's Channel, Colonel Gillespie in his report for 1885 says:

"The borings just made by a diver show that the obstructing shoal is composed of gravel, coarse gray sand and shells for a depth of 2 feet or more, well compacted, underneath which lies coarse sand, the larger shingle of the size of a pigeon egg being on the crest of the bar, and the underlying sand similar to that of the adjacent beach and shoals."

As to the material in the Main Ship Channel, the engineer in his report for 1888 says:

"This latter material is much more difficult to dredge, not only on account of the large percentage of mud too fine to be caught in the bins, but also on account of its lying so compactly on the bottom, and being consequently much more difficult to raise with the pumps."

Difference Between the Amount of Material Removed and Amount Paid For.—Much of the material used in the Main Ship Channel, consisting of fine sand, clay and sedimentary mud, was so nearly of the same specific gravity as water, that when it became agitated and minutely incorporated therewith, by the action of the pumps and currents in the suction pipes and bins of the ships and scows, it settled so slowly in the receiving bins that a portion of it went overboard with the overflow; and, owing to its light weight, was carried by the cross-currents beyond the walls of the channel, greatly to the benefit of the Government and corresponding disadvantage to the contractor. Referring to these transverse currents and their good effect on the desired results of the work performed, Colonel McFarland in his report for 1888 says:

"The work done during the winter on the shoal in the Main Ship Channel was surveyed April 16th, and 177 935 cubic yards measured in place were found to have been removed from the shoals. The quantity removed by the dredges amounted, however, to only 128 453 cubic yards measured in scows, which would not correspond ordinarily to more than 102 762 cubic yards in place. It is apparent, therefore, that the work of the dredges has been materially supplemented by the currents, in fact to the extent of about 73 per cent. The surface currents of the bay at this point run transverse to the channel instead of along its axis, and the tendency is therefore to carry overflow material upon the adjacent shoals. A survey made in June to ascertain whether this material found a lodgment in the channel at some point further down stream, indicates, on the contrary, that the channel has slightly deepened from natural causes alone, both in the prolongation of the dredged area where the work has been done, and in the dredged area itself where work has been suspended for six weeks. These changes are highly satisfactory as far as they go, both as regards the prospects of permanency in the dredged channel and as regards the great saving that will result in the carrying out the project, if through the assistance of the currents the place measurement continues to exceed the scow measurement."

Thus it is seen that on this part of the work, up to the time of this survey, the contractors improved the channel by removing $1\frac{73}{100}$ cubic yards of material for each cubic yard they were paid for handling.

Contracts.—Contracts were awarded at various times to Roy Stone, the Brainards, Joseph Cummings (of the firm of Morris & Cummings), and the Joseph Edwards Dredging Company. The total number of cubic yards excavated from the channels, taken to sea and deposited outside of the Scotland Light-ship, in not less than 14 fathoms of water, was 4 875 079, at a total cost of $1 285 862 94 and an average cost per yard of 26$\frac{4}{10}$ cents. The amount removed by the dredges described in this paper was 4 299 858 yards, at an average of 24$\frac{48}{100}$ cents per yard.

It will also be seen by these tables that the above named company removed this large amount of material at an average price of 24.48 cents per cubic yard; and that the general average price paid to other contractors was 40.53 cents per cubic yard, being at the rate of 65.6 per cent. more than was paid this company.

Details of Contracts with Joseph Edwards Dredging Company.

Date of contract.	Number of yards contracted for.	To be completed.	Extension of time to	Date completed.	Number of yards removed.	Price—cents per cubic yard
Gedney's Channel.						
April 27, 1887...	700 000	Dec. 1, '88.	Dec. 31, '88.	Dec. 22, '88.	770 410	28.5 cents.
Dec. 15, 1888....	600 000	Jan. 1, '90.	....	Nov. 30, '89.	599 362	17. "
Total					1 369 772 yards.	
Average						22.84 cents.
Main Ship Channel.						
May 19, 1887....	1 500 000	Dec. 1, '88.	June 30, '89, and to Dec. 31, '89.	Dec. 3, '89.	1 305 202	28.5 cents.
Mar. 22, 1890...	On Brainard's 1 000 000 yard contract.	June 18, '90, withdrew from the work.	....	....	169 754	16¾ "
Mar. 18, 1890....	425 000	Jan. 1, '91.	....	Aug. 13, '90.	425 000	23.5 "
Aug. 13, 1890....	530 000	June 1, '91.	....	Feb. 6, '91.	530 000	22.6 "
Feb. 16, 1890....	500 000	Oct. 1, '91.	Nov. 1, '89.	Oct. 10, '91.	500 130	23.9 "
Total					2 930 086 yards.	
Average						25.25 cents.

Total Number of Yards and General Average Price.

Gedney's Channel	1 369 772 cubic yards.
Main Ship Channel	2 930 086 " "
Total	4 299 858 cubic yards.

General average price per cubic yard, 24.48 cents.

Details of Contracts with Other Contractors.

Date of contract.	Number of yards contracted for.	To be completed.	Extension of time to	Date completed.	Number of yards removed.	Price per cubic yard.

Gedney's Channel—(Roy Stone).

Date of contract.	Number of yards contracted for.	To be completed.	Extension of time to	Date completed.	Number of yards removed.	Price per cubic yard.
Feb. 7, 1885...	A channel 200 feet wide, 28 feet deep.	June 1, '85.	....	May 14, '85, Mr. Stone was released from contract.	None.	Taken at 33 cents.

Gedney's Channel—(Elijah Brainard).

Date of contract.	Number of yards contracted for.	To be completed.	Extension of time to	Date completed.	Number of yards removed.	Price per cubic yard.
July 31, 1885...	320 000	June 30, '86.	Nov. 1, '86, and to Dec. 31, '86.	Nov., 1886.	303 896	54 cents.

Main Ship Channel—(Brainard Brothers).

Date of contract.	Number of yards contracted for.	To be completed.	Extension of time to	Date completed.	Number of yards removed.	Price per cubic yard.
May 11, 1888...	200 000	Dec. 1, '88.	June 30, '89, and to Dec. 31, '89.	Sept. 30, '89.	200 000	28.5 cents.

Main Ship Channel—(Joseph Cummings).

Date of contract.	Number of yards contracted for.	To be completed.	Extension of time to	Date completed.	Number of yards removed.	Price per cubic yard.
May 11, 1888...	800 000	Dec. 1, '88.	June 30, '89—quantity limited to 200 000 cubic yards.	Withdrew from the work.	None.	Taken at 28.5 cents.

MAIN SHIP CHANNEL—(Brainard Dredging Co.).

Nov. 26, 1889...	1 000 000	Withdrew from the work April 16, 1890—Contract annulled June 18, 1890..............	....	71 325	16¾ cents.

Total... 575 221 yards.
Average... 40.53 cents.

TOTAL NUMBER OF YARDS AND GENERAL AVERAGE PRICE.

Main Ship Channel..	271 325 cubic yards.
Gedney's Channel..	303 896 " "
Total number of yards removed..........................	575 221 cubic yards.

General average price per cubic yard, 40.53 cents.

Work was begun on the first contract September 26th, 1885, and the work on the last contract was completed October 10th, 1891, covering a period of six years and fourteen days.

History of Contracts.—An account of the attempts, failures and relative success of the several contractors will be seen by the following extracts from the annual reports of Colonel McFarland for 1886 and 1887.

"The exposed position and frequent storms on the bar, the great depth of water, and other unfavorable conditions, made it a difficult matter to decide upon the best mode of deepening Gedney's Channel. It was not expected that ordinary clam-shell and dipper dredges could be used there to advantage, and therefore in drawing up the specifications for the work provisos were inserted requiring bidders to furnish plans and descriptions of the plant they proposed to use, and also a proviso that, if after the plant had been in use a reasonable time, and had not obtained good results, the contracts should be annulled, and the contractor should be reimbursed a fair amount for his outlay.

"Bids for this work were opened January 15th, 1885. The lowest bidder was the firm of Morris & Cummings, who proposed to do the work at thirty-five cents per cubic yard, with clam-shell dredges and centrifugal pumps. But apart from the fact that the bid of Morris & Cummings was irregular, there was a bid from Roy Stone, of New York, for deepening the channel by means of what he termed 'hydraulic plowing,' a process which consists of stirring up the material composing the bottom by means of a strong jet of water thrown against the sandy bottom during the ebb tide, which the projector thought was strong enough in Gedney's Channel to carry away the material so loosened.

"As Mr. Stone's offer carried with it no obligation of payment on the part of the Government, unless he should deepen the channel to 28 feet for a width of 200 feet to begin with, and as it was certain that the ordinary means of dredging could not be successfully used in a place so

much exposed to sea action as Gedney's Channel is, it was decided to accept Mr. Stone's bid; but as very little confidence was felt in the success of the process, though it seemed expedient to give it a trial, the time for completing this contract was limited to June 1st, 1885, and, if at that time satisfactory results had not been obtained, the contract was to be annulled without any payment being made to the contractor. A contract to this effect was therefore entered into with Mr. Stone, February 7th, 1885, and on the 24th of March work under it was begun. From the outset the method was unsuccessful, and after a short time, an induction pipe was substituted for the water jets, which was expected to draw the material up to the surface of the water, and thereby give it a better chance of being carried off by the current. This, however, proved no more successful than the first method, and on the 14th of May, 1885, Mr. Stone was released from the contract at his own request. No money was paid him by the Government.

"Major Gillespie immediately wrote to Morris & Cummings to ask if they wished to enter into a contract under their bid of January, but they refused, and the work was advertised May 21st, 1885, and proposals were again opened, June 23d, 1885. The prices of the bids varied from thirty-three cents to $1 50 per cubic yard. Mr. Stone was again the lowest bidder, at thirty-three cents, but from being unable to prove his ability to execute the contract, his bid was thrown out. The second lowest bid was that of the Atlantic Dredging Company, at 47 cents per cubic yard. Their method was not specific, the date of commencement too remote, and the time for the completion of the contract not given. The same may be said of the third lowest bid, that of Michael Murray, at fifty-three cents per cubic yard, except that his individual responsibility did not seem as good as that of the Atlantic Dredging Company. These two bids were therefore thrown out, and the contract was awarded, with the approval of the Chief of Engineers, to the next lowest bidder, Elijah Brainard, at fifty-four cents per cubic yard.

Mr. Brainard proposed to put upon the work, within two weeks of signing the contract, three clam-shell dredges, with a capacity of 4 000 cubic yards per day, and further to build and put on within three months of the date of opening the bids a hydraulic pump excavator, with an estimated capacity of 2 500 yards per day. The contract with Mr. Brainard was signed July 31st, 1885, but as it was found impracticable to keep a scow alongside of the clam-shell dredges in the rough water on the bar, it was not thought advisable to attempt to put them to work. On the 26th of September, 1885, the hydraulic excavator was ready and began work. It was expected by the contractor that this machine would raise from 10 to 20 per cent. of solid material, which would correspond to 2 000 to 3 000 cubic yards per day. The amount actually raised was less than 5 per cent.

Beginning September 26th, 1885, only fifty-seven hours and forty minutes actual pumping was done up to December 23d, 1885, when, in a storm, going out to the dumping ground, the A-frames, derrick-frames and suction pipe were carried away, and the excavator had to be laid up for repairs. In the fifty-seven hours forty minutes pumping, 6 411 cubic yards of solid material were excavated. The repairs consisted of replacing the old engine with a new one of double power, the derrick-frames were rebuilt and strengthened, the suction pipe was reinforced with a truss, condensers were put in, and various minor repairs and improvements were made, and the excavator was ready for work again February 24th, 1886. Meanwhile, under date of January 2d, 1886, Major Gillespie notified the contractor by letter that the plant did not come up to the capacity called for by the specifications."

Mr. Brainard replied that it was his intention soon to work additional plant of much greater capacity, and that with the experience already had, he anticipated excavating with the new plant from 3 000 to 3 500 cubic yards per diem.

"The excavator was not finally finished until May 29th, 1886. The hull was especially built for the purpose; its dimensions are 152 feet by 44 feet, and it draws about 16 feet loaded; it is scow built, with six pockets and bottom dump valves. The capacity of the pump, as estimated by its makers, is 36 000 gallons per minute, through a 22-inch suction and a 22-inch discharge pipe. Her loaded capacity is 1 055 cubic yards when full. She is not self-propelling, but is towed.

"The *Howard*, another barge which was working last season on Charleston Bar, is self-propelling, and is about 110 feet by 32 feet, and draws about 10 feet. An Edwards pump, capacity unknown, is set low amidships. There are two suctions, one on each side, 9 inches in diameter. The load is carried on an upper deck forward, in bins, which dump through side doors. Her full loaded capacity is about 175 cubic yards.

"The following is a statement of the work of the three excavators since they commenced work under this contract:

	Cubic Yards.
Excavator No. 1	63 935.0
Excavator No. 2	44 684.7
Excavator *Howard*	29 581.4
Total for about 1 year's work to July 31, 1886	138 201.1

"The contract had to be extended, and the work was finally completed and the contract was closed early in November, 1886. The amount of material removed was 303 869 cubic yards."

When it was concluded by the Government that the entire improvement was to be made by dredging, it was found that contractors hesitated

to enter upon the undertaking, as will best be seen by the following account taken from Colonel McFarland's report for 1887:

"In accordance with the recommendation of the Board of Engineers approved by the Secretary of War December 27th, 1886, under date of January 22d, 1887, proposals for deepening Gedney's Channel were called for by public advertisement, the bids to be opened February 23d, 1887. Proposals for deepening the Main Ship Channel were to be opened at the same time, under an advertisement dated January 14th, asking for them. The result of the bidding, however, was not satisfactory. Only one bid was received for the work on the Main Ship Channel, and as this proposed the use of experimental plant, which was forbidden by the specification, it was rejected. Five bids were received for the Gedney's Channel work, but as it seems likely, in view of the failure of the contractors to bid upon the Main Ship Channel work, which was much the larger part, that the Government might have to build its own plant and to do the work itself, it was deemed inexpedient to let a part of it only, and the bids for Gedney's Channel were rejected also."

In view of this situation of affairs, the report goes on to say:

"A circular letter was issued by Colonel McFarland and sent not only to every one who had previously bid upon the work, but to every one to whom the original specifications had been sent, and to others besides, asking them to state under what circumstances, at what rates and in what time they would undertake to do the work of deepening either or both the Main Ship Channel and Gedney's Channel. That circular was as follows:"

"Gentlemen,—It is evident from the absence of bids for dredging the Main Ship Channel, New York Harbor, asked for under my advertisement of January 22d, 1887, that there must have been some limitation in the published specifications which those who are accustomed to doing that kind of work were unwilling to accept.

"With the view of getting the work done as speedily and cheaply as possible, I should be glad to have those who are engaged in the business of dredging, or who are accustomed to the management of large contracts, communicate with me in writing, stating under what circumstances and at what rates and in what time they would be willing to undertake and to complete the deepening of either or both the Main Ship Channel and Gedney's Channel, New York Harbor, the total amount to be expended on both of these improvements being about $650 000, and the amount of material to be removed being between 2 000 000 and 3 000 000 cubic yards.

"It is possible that some offer may be made which would be acceptable and which would render it unnecessary for the Government to undertake to do the work itself.

"A free expression of views is invited. As it is important to have the work done as soon as possible, it is desirable to have the matter settled by the 10th of March."

"In answer to this circular ten communications were received, some

for doing the whole work, some for the Main Ship Channel alone, and some for Gedney's Channel alone, all varying in rates, time and method of doing the work. None of the offers for the whole work were entirely satisfactory, however; for although some of the rates offered were low, the time for the completion of the work was too great; but for the Gedney's Channel work, for which only six offers were made, a lower bid than any previously offered was received, and by authority of the Secretary of War it was accepted, and an agreement in accordance with it, dated April 21st, 1887, was made with the Joseph Edwards Dredging Company for removing 700 000 cubic yards of material from Gedney's Channel, at twenty-eight and a half cents per cubic yard. This was six and a quarter cents a yard lower than the lowest offer previously received under advertisement of January 22d, 1887." Two offers were also received for the Main Ship Channel work, but as one of these was for a part of it only, and the other involved the use of experimental plant, both were rejected.

"On the 22d of April, the Joseph Edwards Dredging Company made a proposal in writing to do the Main Ship Channel work at the same rate at which they agreed to do the Gedney's Channel work, namely, at twenty-eight and one-half cents per cubic yard, suggesting at the same time that if their offer were accepted, the additional plant designed for working in the Main Ship Channel might, if desired, be united with that intended for Gedney's Channel, and the work upon that channel might then be completed before the close of the present year; and afterward the combined plant working upon the Main Ship Channel could finish the deepening of that channel within the succeeding year, the period fixed for its completion by the specifications previously issued. A copy of this letter accompanies this report.

"This offer was so favorable, so much time had already been lost, and the deepening of Gedney's Channel before another winter should set in was so very desirable in an engineering point of view and so impossible of accomplishment in any other way than that proposed, that the offer was immediately forwarded to Washington, with a strong recommendation that it be accepted; but it was decided by the Secretary of War that this could not be done, and that the work must be advertised again. This was accordingly done under date of May 3, and bids were opened May 18, 1887.

"Only two were received, and the lowest of these was from the Joseph Edwards Dredging Company, the rate being the same as in the previous offer. This offer was accepted, and a contract was entered into with them, dated May 19th, 1887, for removing 1 500 000 cubic yards of material from the Main Ship or the Swash Channel, as might be found most expedient, at twenty-eight and one-half cents per cubic yard.

"The whole work, therefore, of improving Gedney's Channel and the Main Ship Channel is now in the hands of this one firm, the manag-

ing head, Mr. Edwards, having had large experience with similar dredging methods on ocean bars, under General Gilmore, at the mouth of the St. John's River, Florida, in 1871, and subsequently under Captain Eads in his operations at the mouth of the Mississippi River.

"The plant to be used on the two channels will consist of three self-propelling hydraulic dredges, of a combined daily capacity of 10 000 cubic yards or over. The first one, the *Bolivar*" (subsequently named *Advance*), "which is to be ready July 1st, is a steam vessel, furnished with two 15-inch centrifugal pumps, and capable of carrying a load of 450 cubic yards. The second steamer, the *Mount Waldo*, which is to be ready August 1st, is to have two 15-inch centrifugal pumps and a carrying capacity of 350 cubic yards, and is to be accompanied by four scows, each of 500 cubic yards capacity, each towed by its own tug. The third dredge, the *Reliance*, which is to be ready September 1st, is a new steamer, built for the purpose, and is to have two 8-inch centrifugal pumps and a carrying capacity of 650 cubic yards."

As appears by the engineer's report for 1889: "On December 8th, 1888, a proposition was received from the Joseph Edwards Dredging Company to dredge from the Gedney's Channel division about 600 000 cubic yards, which, it was estimated, would complete the improvement projected for that division, at seventeen cents per cubic yard, in accordance with the specifications governing the execution of their contract which had just expired, with the reservation that no deduction should be made for over-depths of less than 1 foot from the payment due for work done under the agreement. This proposition was approved by the Chief of Engineers December 15th, 1888, and an agreement was accordingly made, with the limitation that the work should be completed by January 1st, 1890."

As appears by the Engineer's report for 1890:

"Proposals were invited by public advertisement and opened March 13th, 1890, for the removal of 500 000 cubic yards of material from the Main Ship Channel north of Buoy No. 12. But one bid, presented by the Joseph Edwards Dredging Company, at 23.5 cents per cubic yard, was received, and being considered reasonable, a contract was entered into with this company March 18th, 1890, approved by the Chief of Engineers Aprill 11th, 1890, for the removal of 425 000 cubic yards.

"Operations under this contract began with the dredge *Advance* April 2d, with the dredge *Reliance* April 4th, and with the dredge *Mount Waldo* June 18th, 1890; and the work was finished August 13th, 1890. On the same date the company contracted to remove 530 000 cubic yards from the Main Ship Channel at 22.6 cents per cubic yard. Under this contract the number of yards removed was the same as the amount called for; and the work was completed February 6th, 1891. February 16th, 1891, a contract was awarded the same company for the removal of 500 000 cubic yards from the Main Ship Channel at 23.9 cents per cubic

yard. Under this contract the number of yards of material removed was 500 130, and the work was finished October 10th, 1891; and with the completion of this contract was completed the improvement of New York Harbor, the company having executed about 88 per cent of the work."

It is not necessary to enter into the details of the other contracts, as but little was accomplished by the contractors.

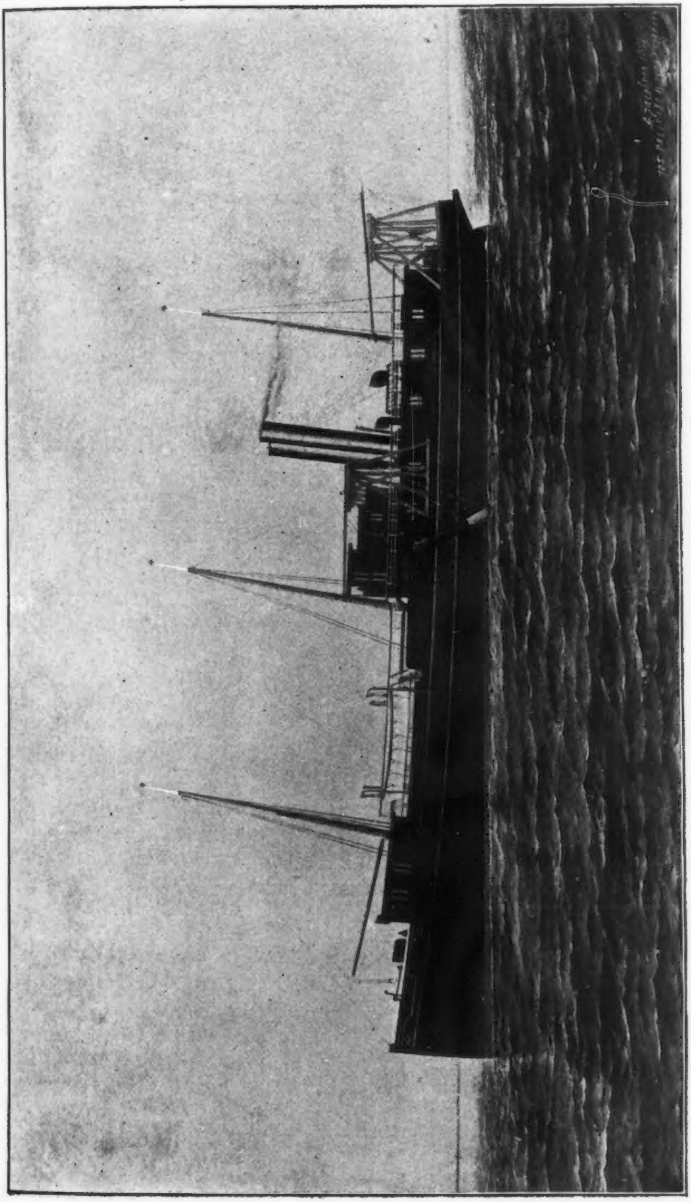
The magnitude of the undertaking will be seen by considering that besides the excavation of about 5 000 000 cubic yards of material and its transportation out to sea (the average round trip being 21 miles), there had to be dredged, as before explained, a large amount more which went overboard with the overflow, especially in removing the crust of mud and clay in the Main Ship Channel. This material had to be raised from a depth of 24 to 35 feet under water and elevated to a height on shipboard which, from the bed of the channels, amounted to an elevation of from 36 to 46 feet, according to the state of the tide and the depth of the channel. For each cubic yard of solid material thus handled there had to be raised many cubic yards of water, at a height, from the surface of the bay to the mouth of the discharge pipes, of from 10 to 15 feet. The mixture of mud, clay, sand and water thus raised amounted to many times the cubic yards of material which the contractor was paid for handling.

System of Dredging Employed.—The system of dredging employed was what is known as that form of hydraulic dredging in which the means for excavating and elevating the material to be removed are centrifugal pumps, illustrations of those employed being shown in Figs. 1 to 5.

But besides the employment of centrifugal pumps, it was necessary to devise the plant in such a manner as to adapt it to the peculiarities of the work, having reference to the exposed position, depth of water, etc.

The plant provided for executing the work consisted of three seagoing dredging steamers, four large scows, four steam tugs for towing the scows, one steam supply boat, one steam tender, docks, repair shops, storehouses, water-works, coal bins, etc. The dredging steamers, known as the *Reliance* (see Plate CIV), the *Advance* and *Mount Waldo*, are not essentially unlike other sea-going steamers, aside from their dredging outfits. Their dimensions and daily working capacity are as follows:

PLATE CIV.
TRANS. AM. SOC. C. E.
VOL. XXV, No. 513
EDWARDS ON DREDGING NEW YORK HARBOR



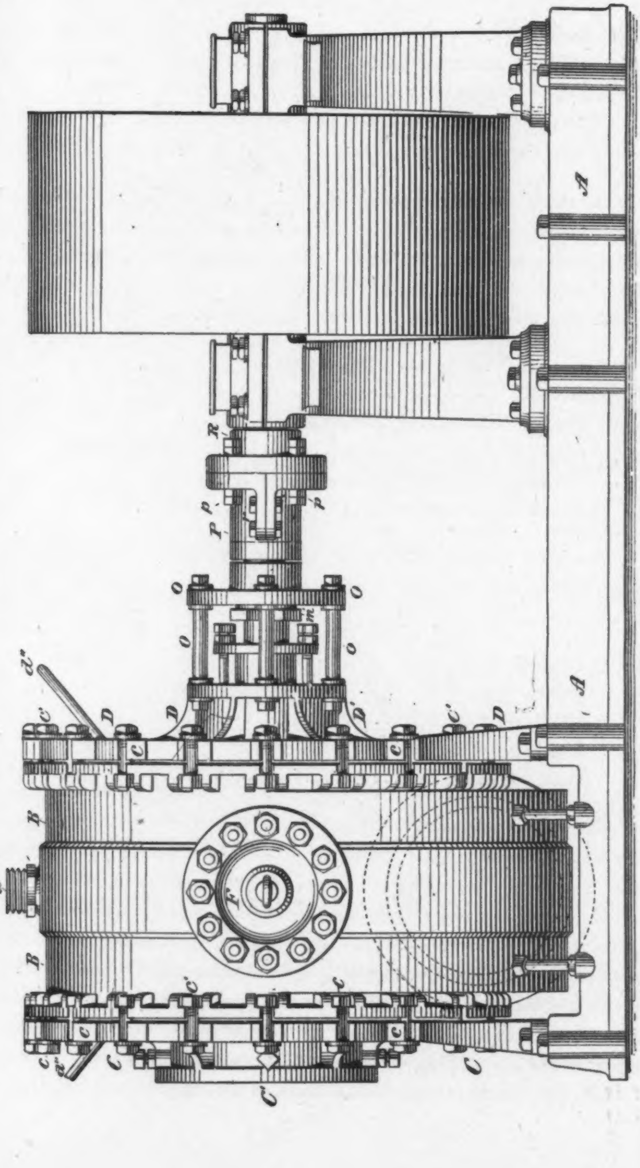


Fig. 1.—Exterior View of Edwards' Cataract Pump.

THE *Reliance.*

Length .... 157 feet.
Beam....................................... 37 "
Depth of hold.............................. 16 "
Twin propellers and compound engines.
Carrying capacity.................... 650 cubic yards.

The daily working capacity of this dredge on Gedney's Channel, where the material consisted of coarse sand, and freighting it about six miles each way, was seven loads; and on the Main Ship Channel, where the material consisted of mud, clay and fine sand, and freighting it a distance of about 12 miles each way, was three loads.

THE *Advance.*

Length.................................... 132 feet
Beam...................................... 34 "
Depth of hold............................. 8 "
Single propeller and compound engine.
Carrying capacity.................... 500 cubic yards.

The daily working capacity of this dredge, on the same channels, respectively, was also seven loads and three loads.

THE *Mount Waldo.*

Length.................................... 145 feet.
Beam...................................... 31 "
Depth of hold............................. 11 "
Single propeller and compound engine.
Carrying capacity.................... 275 cubic yards.

The daily working capacity of this dredge, discharging into scows, working on the Main Ship Channel, on material consisting of mud, clay and fine sand, was eight scowloads of 500 yards each. The *Mount Waldo,* except in the winter-time, instead of being employed as the other two steamers were, namely, for dredging and transportation, was kept constantly employed at dredging by discharging the material into scows and not into her own bins; the scows were towed to the dumping ground by steam tugs. Besides the three above described steamer dredges and four scows, there were four powerful sea-going tugs for towing the scows to sea; also a large steam lighter employed as a

supply boat for furnishing the dredging steamers with coal, water, duplicate parts, provisions, etc.; and also a steam tug employed as a general tender for convenience of the General Manager and Superintendent.

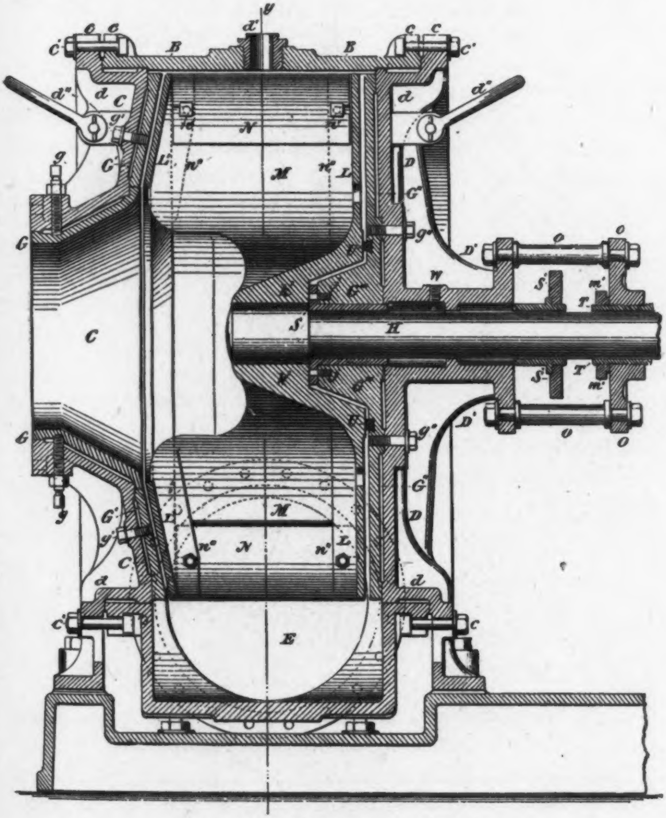


FIG. 2.—Central Vertical Section of Pump.

Each steam dredge was provided with two pumping outfits independently arranged, so that either could be operated regardless of the other; each outfit having a centrifugal pump, engine, suction and discharge pipes. The suction pipes, corresponding to the size of the pumps, one on either side of the steamer, were located about mid-

ships, and extend laterally from the pumps to the outside of the ship; then they turn with an easy bend at right angles, extend (when not in use) along the sides of the dredge, and are held up, lowered and raised by suitable hoisting purchases worked by hoisting engines.

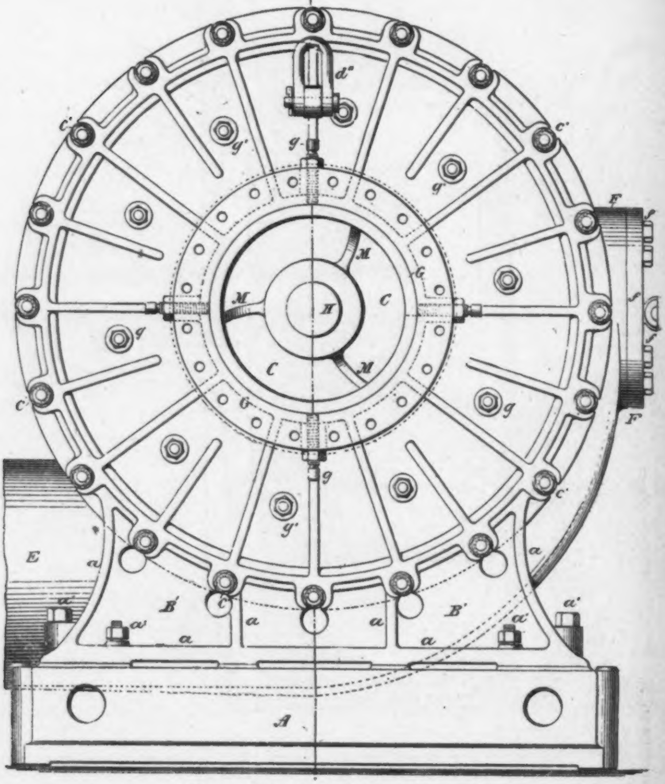


FIG. 3.—View of Suction End of Pumps.

These suction pipes, 15 and 18 inches in diameter, are about 60 feet long, terminating about opposite the stern of the ship, with suitable mouth pieces, termed drags, to fit on the bed of the channel and facilitate the ingress of the material. (See Plates CV, CVI, CVII, CVIII.)

To render the suction pipes flexible, so as to accommodate them-

selves to the pitching and rolling motions of the steamer, a section of them about 12 feet in length, located a few feet from the bend, consists of rubber; these flexible sections being supported by a special hanging arrangement of chains, blocks and hoisting purchases against the vertical strain caused by the weight of the suction pipes themselves and

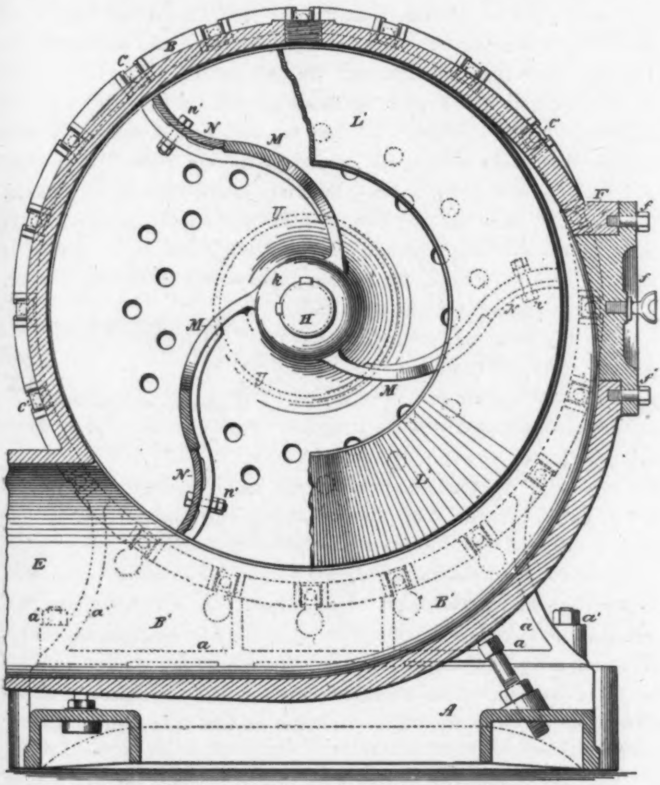


FIG. 4.—Interior View of the Pump, Showing Detachable Plates in Place.

what passes through them; and by tension chains against the longitudinal strain of the drags resting on the bottom. The steamers and scows were provided with special arrangements to afford a long flow of the mixture of mud, clay, sand and water, between the discharge from the pumps and the overflow outlets, to facilitate the settlement of the solid material in their bins.

The scows were divided into compartments surmounted with longitudinal sluice-ways extending either way from a central receiving hopper. These sluices were provided along their course with a series of adjustable bottom and side gates by means of which the material could be deposited faster or slower in the different compartments; whereby the load could be uniformly distributed throughout the length and breadth of the scows, to prevent them from listing and enable them to be freighted to their full capacity. (See Plates CIX, CX.)

The steamers were kept under headway from the time they left their anchorage in the morning until they returned to it at night. When a dredge reached the channel the suction pipes were lowered to an angle of from 30 to 40 degrees, to bring the drags in contact with the bottom. To keep the vessel on her line of work and supply the drags with material, she was kept constantly under steering headway. As soon as the sand bins were filled, the suction pipes were hoisted out of water, and the steamer put under full headway for the dumping ground. While she was turning to return to the work again, the dump gates were opened and her cargo discharged—the discharge being facilitated by pumping water into the bins with the dredging pumps. On again reaching the work her speed was slackened, drags lowered and pumps started—and so on, until the time to return to her anchorage for the night.

FIG. 5.—Detachable Plate at Outer End of Pump Vanes.

The above brief general description affords but a limited view of the construction and performance of the plant. To fully appreciate the arrangement and effective execution of the steamer dredges, they need to be seen in operation, especially in a heavy seaway. Some idea may be had of the extent of the plant from the fact that in connection therewith in one way and another, it included no less than eighty different steam cylinders.

Difficulties Encountered.—The lower bay, in which the Main Ship Channel is located, embraces an area of about 100 square miles, estimating inside, northward and westward, of a line drawn from Sandy Hook to Coney Island, with an opening outward to the ocean (between Sandy Hook and Coney Island) 7 miles wide. Through this opening, the work on the Main Ship Channel was exposed to the full sweep of the ocean from the northward around by the east to the southeast, and from the

southwest and west to winds sweeping down Raritan Bay, while winds from the northwest and north had scope across and down the channel from Staten Island and the Narrows. Thus exposed to the wind from every quarter, except the south, it was often impossible to handle and tow the scows or even to work the dredging steamers, owing to the roughness of the bay, even in clear weather.

The Gedney's Channel being outside of the entrance to the bay, may be considered as located in the Atlantic itself, and therefore the work on this channel was exposed to the winds and roughness of the ocean from all directions, making it impossible to work here even as constantly as on the Main Ship Channel; while it was impossible to work on either channel during an easterly storm, and sometimes for days after, owing to the continuation of an incoming rolling sea; and often when the dredges did work they were more or less belabored with rough seas.

The dumping ground being outside of the Scotland Light-ship, the material removed from the Main Ship Channel had to be transported an average distance of 12 to 14 miles, making the round trip 26 to 28 miles, and that from the Gedney's Channel 6 to 7 miles, making the round trip 12 to 14 miles. As regards the transportation by towing, there was nearly always sufficient sea on to more or less strain the scows, and require a powerful tug to handle each separate scow.

To avoid liability of collision with passing vessels, especially with large ocean steamers, it was frequently necessary to shift the position and stop the work of the dredges until the steamers had passed. Besides this, these larger steamers produced a disturbing wake of the water, and passing so frequently, interfered not a little with the work. Notwithstanding the utmost caution, two collisions occurred, and one of the company's ships, the *Advance*, was run down, sunk and destroyed during the progress of the work.

On account of rough sea, heavy storms and otherwise inclement weather, as also on account of thick and foggy atmosphere, even though there were no storm or roughness of sea, and sometimes because of running ice, it was often necessary to suspend operations of the entire plant from one to several days with no diminution of expense save a trifle on fuel; such delays hindered the progress of the work and diminished the chances of profit to the company.

As might be expected, in channels so long and extensively navigated, there were frequently found minor yet troublesome obstructions, as

fragments of wrecks, anchors, chains, bars of iron, cannon balls, etc., which when encountered sometimes caused damage to the pumps, suction pipes and drags. Such objects, encountered and passed over, would again and again be found in the way, necessitating a search for them and their removal. Sometimes before being definitely located and removed, they would not only break a drag, but carry away both drag and suction pipe, and delay the operation of the entire dredge for some days, resulting in a loss to the contractor of several thousand dollars.

The constant transverse currents that set nearly at right angles across that portion of the Main Ship Channel extending north and south, necessitated the heading of the dredging ships in a diagonal direction to the line of the channel, thus causing the cross currents to carry one of the suction pipes with its drag or mouth away from the side, and the other under the bottom of the dredging steamers. This was a serious and constant interference with their most successful operation, and frequently caused injury to the suction pipes, sometimes badly breaking them.

The composition of a part of the material in the Main Ship Channel was of such a nature, being nearly of the same specific gravity as water, that, when it became thoroughly agitated and incorporated with the large amount of water handled, only a variable percentage of the amount dredged would settle in the scows and bins of the steamers, while the balance of it would go overboard with the overflow—the respective proportions that would settle and overflow depending, of course, on the nature of the material being dredged.

The extent and nature of the plant, and the character and location of the work performed, rendered it impossible to avoid numerous accidents and break downs, thus necessitating the keeping on hand of a large supply of duplicate parts to diminish delay of repairs, but which greatly increased the cost of the general outfit.

Owing to the tension to which the mechanism of the plant was submitted, and the speed at which it was driven, the cutting effect of sand rapidly forced through the suction pipes and pumps and scattered by wind and spray throughout the machinery, and the strain upon the ships incident to being constantly loaded and unloaded,—the extensive wear and depreciation of the plant constituted a serious drawback to the contractor, it becoming necessary for the outfit to

undergo general yearly repairs amounting sometimes to a cost of fifty thousand dollars.

On pages 606 and 607 is a statement of the work done by the *Reliance* in Gedney's Channel in twenty-eight and one-half working days in September and October 1891, taken from the daily records:

Average time pumping per load	$48\frac{6}{10}$ minutes.
" cubic yards per load	584.87 cubic yards.
" time pumping per day	4 hours $58\frac{4}{10}$ minutes.
" " on bar per day	5 hours $43\frac{4}{10}$ minutes.
" " bar to dump	34 minutes.
" " dumping	$12\frac{1}{10}$ minutes.
" " dump to bar	$25\frac{7}{10}$ minutes.
" " " to anchor	$66\frac{6}{10}$ minutes.
" " anchor to bar	51 minutes.
" " under steam per day	16 hours $4\frac{2}{10}$ minutes.
" No. of loads per day worked	6.73
" cubic yards per day worked	3 936.65 cubic yards.
" rate per minute	12.03 " "
Time lost by repairs	2 hours 24 minutes.
" " " weather	32 " 50 "
Total time lost	35 " 14 "

in a total of 458 working hours in 28½ working days.

September 10th, 4 790 yards, 8 loads, or at the rate of 16.43 cubic yards per minute. The 6th load of 611 yards was done at the rate of 15.27 yards per minute, and the 7th load of 608 yards was done at the rate of 16.43 yards per minute or an average of 946 cubic feet per hour.

September 11th, 4 582 yards, 7 loads, or at the rate of 15.27 cubic yards per minute.

Government Engineers in Charge.—The engineers under whose direction the improvement of New York Harbor was conducted were Major (subsequently Lieutenant-Colonel) George L. Gillespie and Lieutenant-Colonel Walter McFarland, Corps of Engineers, U. S. Army. Colonel Gillespie being in charge at the commencement, made the preliminary survey, and projected and recommended the plan by which the improvement has been expeditiously, successfully and satisfactorily accomplished, and at a cost to the Government not exceeding 25 per cent. of

Date.	Cubic yards excavated.	Time pumping.	Rate per minute.	Total time on bar.	Time Lost. Turning.	Bar to dump.	Dumping.	Dump to bar.	Dump to anchor.	Off cut and other causes.	Anchor to bar.	By repairs.	By weather.	Total time.	Direction and force of wind.	Condition of sea.	No. of loads.	Remarks.
		H. M.	Cubic yards	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M.	H. M				
1891.																		
Aug. 27	2 150	3—08	11.44	3—29	0—10	2—15	0—56	1—12	1—05	0—11	......	......	......	8—58	S. E. to S. S. W.	Smooth to Chop	4	One load from Main Ship Channel in the morning.
28	4 174	5—28	12.72	6—04	0—18	3—51	1—36	2—23	1—12	0—18	0—47	......	......	16—25	S. W. to N. W.	Chop	7	
29	4 196	5—43	12.23	6—22	0—33	3—44	1—27	2—32	1—17	0—06	0—38	......	......	16—00	N. W. to S. W.	Smooth	7	Week ending Aug. 29th, 3½ days on Main Ship Channel.
31	1 666	2—20	11.82	2—37	0—11	1—26	0—39	0—43	1—25	8—04	1—02	......	......	15—50	Fresh. S. E.	Swell.	3	
Sept. 1	4 563	5—42	13.34	6—44	0—26	3—54	1—31	2—45	0—55	0—36	1—08	......	......	16—57	Light. N. N. E.	Swell.	8	
2	4 344	5—50	12.40	7—03	0—32	3—49	1—19	2—41	0—52	0—41	1—07	......	......	16—51	Light. N. W.	Smooth	7	Foggy in the morning.
3	4 376	6—17	11.60	7—16	0—31	3—46	1—20	2—14	0—52	0—28	0—43	......	......	16—11	Light. W. S. W.	Smooth	7	
4	4 341	6—21	11.40	7—27	0—30	4—12	1—25	2—03	0—54	0—36	0—42	......	......	16—43	Fresh E. S. E.	Rough.	7	
5	1 368	1—41	13.54	2—10	0—14	1—38	1—32	0—42	1—46	0—15	0—54	......	8—58	17—40	Strong N. E.	Very Rough	3	Too rough to work.
7	4 056	6—04	11.14	6—58	0—18	3—57	1—24	2—34	0—56	0—36	1—06	......	......	16—55	Light. N. W.	Chop.	7	
8	4 046	5—35	12.08	6—17	0—21	3—53	1—36	2—28	1—00	0—21	0—32	0—58	......	16—44	N. W.	Smooth	7	Valve-stem of large washout pump broke.
9	3 148	5—06	10.28	7—32	0—26	2—57	1—23	2—18	0—58	0—34	0—52	1—26	......	16—00	N. W.	Smooth	6	Belt on port pump broke.

	10	4 790	6—24	12.47	7—13	0—30	3—53	1—37	2—42	1—02	0—19	0—45	......	......	17—12	N. W.	Calm.	8	Sixth load (of 611 cubic yards) was at the rate of 15.27 cubic yards per minute.
	11	4 582	6—10	12.38	7—17	0—34	3—55	1—29	2—34	1—02	0—23	0—33	......	......	16—40	Light. N. W. to S. W.	Calm to Chop.	8	Seventh load (of 608 cubic yards) was at the rate of 16.43 cubic yards per minute.
	12	4 055	6—33	10.32	7—47	0—37	3—30	1—27	2—09	1—27	0—37	0—33	......	......	16—53	Brisk. S. E.	Chop.	7	
	14	3 510	6—16	9.33	7—20	0—39	3—25	1—18	2—29	1—19	0—25	1—11	......	......	17—02	Brisk. N.	Heavy Chop.	7	
	15	4 076	6—30	10.45	7—43	0—30	3—38	1—24	2—16	1—06	0—43	0—48	......	......	16—55	Light. N. W.	Smooth	7	
	16	4 112	6—26	10.65	7—26	0—37	3—26	1—25	2—06	1—12	0—23	0—51	......	......	16—25	Light. N. W.	Smooth	7	
	17	4 296	4—44	15.12	5—17	0—16	4—43	1—13	2—59	0—58	0—17	1—00	......	......	16—10	Fresh. S. S W.	Chop.	7	
	18	4 232	4—46	14.80	5—03	0—09	4—39	1—16	2—59	0—53	0—08	1—01	......	......	15—51	Light. N. W.	Chop.	7	
	19	3 630	4—51	12.47	5—55	0—21	3—58	1—10	2—29	1—13	0—43	1—00	......	......	15—45	Light. V'riable	Smooth	6	September 28th, 29th and 30th, worked on the Main Ship Channel.
Oct.	1	1 761	2—26	12.06	2—26	......	2—23	1—02	0—58	1—02	......	......	......	......	7—51	Stormy. E.	Rough.	3	One load on Main Ship Channel. Started for Gedney's Channel 12.09 P. M.
	2	3 682	4—18	14.27	4—51	0—10	4—09	1—14	2—46	0—58	0—23	0—46	......	......	14—44	Light. V'riable	Swell.	6	
	3	3 069	4—12	12.13	4—22	0—04	3—30	1—02	2—06	1—18	0—06	0—50	......	0—50	13—58	High. S. E.	Swell.	5	
	5	2 370	3—17	12.03	3—24	0—07	2—40	0—48	1—35	1—06	0—20	1—00	......	4—00	14—53	Light to Strong. S. E.	Heavy. Swell.	4	Delayed at anchor by fog.
	6	3 766	4—29	14.00	5—04	0—16	3—55	1—04	2—43	0—54	0—19	0—35	......	......	14—15	Fresh. N. W. to N. E.	Chop	6	Oct. 7th, S. E. storm—wind and rain—no work.
	8	2 088	3—11	10.93	3—35	0—19	2—29	0—42	1—48	1—03	0—35	0—48	......	4—00	14—55	Fresh. N. W.	Heavy. Swell.	4	Delayed at anchor by weather.
	9	3 690	4—44	13.00	4—59	0—15	3—31	1—07	2—12	1—28	......	0—46	......	......	14—03	High. N	Swell.	6	
	10	2 216	3—14	11.42	3—26	0—08	2—17	0—56	1—09	1—00	0—04	0—57	......	......	9—45	High. N	Swell.	4	
		102 353	141—46	12.03	163—07	10—02	99—22	30—22	62—35	32—13	18—31	22—55	2—24	17—48	448—00	....	....	175	

the amount estimated to be necessary to attain the desired results by the only other project presented. During an absence of Colonel Gillespie the improvement was in charge of Colonel McFarland. After his demise Colonel Gillespie again took charge, and under him, and in accordance with his plan and recommendation, the improvement was completed.

As was to be expected, in an undertaking of such a nature and magnitude as this improvement, various difficulties had to be encountered and overcome, by the engineers in charge as well as the contractors; in view of which it is worthy to mention that, from the commencement to completion of the work, the most harmonious relations existed between them; the representatives of the Government being always considerate of the contractors, they, in turn, co-operating to bring the undertaking to a successful consummation.

The Pumps.—As the pumps are the chief features of the plant, it can be readily understood that, substantially, as work the pumps so works the plant. Though a self-propelling and self-containing dredging steamer needs to be in all its appointments rightly constructed and properly managed, yet it is apparent that however well everything else may be contrived, and however skillfully managed, the results attained will be but partial without an effective pump. Besides, as the pumps are the parts of the plant submitted to the greatest wear by the action of sand, and are liable to the greatest number of breakages from various resistant objects passing through them, they need to be provided with the best possible means of endurance and ready repair. That the pumps employed on this work meet all these requirements is sufficiently demonstrated by what has been done with them, not only on the improvement of New York Harbor, but by what they have accomplished on difficult work elsewhere. So well contrived to resist breakage and so powerful are they, that they have drawn up from a depth of 35 feet under water, and elevated, from the bed of the channel to the mouth of the discharge pipe, a height of 46 feet, bars of pig iron heavy as a man can lift, cannon balls, etc., and with no damage to themselves requiring more than twenty minutes to repair. Without special provisions for encountering such obstacles, they would often have totally wrecked the pumps.

Pump Dredging.—Pump dredging can be employed where no other system of dredging is possible, as shown by its application to the improve-

ment of New York Harbor, on which the steamer dredges have successfully worked in a heavy seaway. It leaves a smoother and more uniform bottom than any other system. It can do the work at much less cost than any other system of dredging, wherever it is applicable. It can go over the ground much more rapidly where only a slight cut of material is required to be taken, and it is the only system of dredging that can be employed to keep channels open that have a tendency to fill up, as a dredging steamer can pass over large areas and take only the needed skimming of material.

Reasons for Improving Other Channels.—As the improvement by widening and deepening Gedney's and Main Ship channels has been successfully accomplished at an expense to the Government not exceeding 23¼ per cent. of the originally estimated cost, and as the improvement of these channels is proven to be permanent, more and better reasons than before now present themselves to show why the Swash Channel also should be improved.

The Permanence of these Improvements.—When the Government concluded to make appropriations for the improvement of New York Harbor, so much doubt existed on the part of its engineers as to whether it could be accomplished by dredging, both because of the uncertainty of the permanency of the work and the difficulty of its execution, that they concluded the best project for accomplishing the improvement of Gedney's Channel and that portion of the Main Ship Channel lying east and west, would be to contract the flow of water between Coney Island and Sandy Hook, by constructing a half-tide or higher stone dike from Coney Island, 4 miles or more toward Sandy Hook; and to dredge that part of the Main Ship Channel lying north and south, at an estimated cost of from \$5 000 000 to \$6 000 000.

Owing, however, to the first appropriation (\$200 000) being stipulated for dredging in Gedney's Channel, dredging in this channel was commenced, but with little expectation on the part of the Board of Engineers that the results would be permanent. Careful comparative surveys made at a later time to determine the question of permanency, showed that so far as the work had been done the depth of the channel was fully maintained, whereupon dredging was continued. Other surveys, for the same purpose, from time to time, were made, which further confirmed the same satisfactory results.

In every instance test surveys show that the increased depths of the

channels are fully maintained and even increased. Hence it is safe to conclude that the Swash Channel, were it similarly improved by the same means, would also maintain its depth and width. Other reasons for this conclusion are:

First.—The Swash is not exposed to sedimentary deposits from the Raritan Bay as is the lower portion of the Main Ship Channel.

Second.—It is a shorter channel-distance from the upper part of the Main Ship Channel to the ocean, and, therefore, would be more likely to be scoured by the water from the Narrows, by way of the upper part of the Main Ship Channel than would be the lower section of the latter channel.

Third.—It is a more direct course between the upper part of the Main Ship and the Gedney's channels than is the Main Ship from its upper union with the Swash to the Gedney's Channel.

Fourth.—Surveys of the various channels from as far back as 1835 show that the Swash is in as good condition as it was 56 years ago.

Fifth.—The velocity of the flow of water through the Swash is twice as great as it is in the Main Ship Channel on the Knolls, and its flow is nearly parallel with its axis (the same is true of the Gedney's Channel), which also facilitates navigation, while part of the tide crosses the axis of the Main Ship Channel.

In confirmation of these statements the following is taken from Colonel McFarland's report for 1886:

"A comparison of the survey made by Major G. L. Gillespie, Corps of Engineers, in 1884 (see annual Report of the Chief of Enginers for 1885, pages 776 to 785) with those previously made, shows that the changes which have occurred in the channels since 1835 have been very slight. The 24-foot curves of the Main, Swash, East and Fourteen-Foot channels have all moved slightly westward and southward, but still embrace about the same area of shoals. But while, in 1835, bars with less than 24 feet on them extended completely across Gedney's, the Main Ship and the Swash channels, these bars, at the time of Colonel Gillespie's survey in 1884, had nearly disappeared, leaving in each case only spots where there was less than 24 feet of water. For fifty years, then, it will be seen that the natural tidal scour, which is strong enough at the Narrows to maintain a channel a mile wide, with depths of 100 feet in it, has been only able below the Narrows to maintain a 30-foot channel, with two spots in it where there is 24 feet depth.

"From current observations taken by Colonel Gillespie in 1884 it appears that a good deal of the ebb volume from the Narrows passes to the northward of the south side of the Swash Channel and that a large

part of the Raritan Bay ebb volume passes between the south side of the Knolls and Sandy Hook, leaving a wedge-shaped area at their junction near the Knolls, over which the rate of the current is reduced about one-half and where a part of the current passes directly across the channel instead of along its axis.

"This is also true of the flood tide; on the flood there is the same diminution of rate across this area, and a part of the tide crosses the channel at right angles. Some of the velocities observed by Colonel Gillespie in 1884 in these channels are as follows:

LOCALITIES.	Maximum ebb (miles per hour).	Maximum flood (miles per hour).
In the Swash Channel, abreast of the Roemer Beacon......	1.60	1.20
In Main Ship Channel, on the Knolls........................	0.80	0.65
Close off the point of Sandy Hook, and due north of it.....	2.22	1.70 to 1.20
In the deep hole half way between Sandy Hook and Gedney's	2.00	1.50
In Gedney's Channel..	1.60	1.30

"Both in the Swash and in Gedney's Channel the ebb current is nearly parallel with the axis of the channels, and from the measurements so far taken the relative ebb velocities in both channels appear to be the same, but the flood in Gedney's Channel appears to exceed that of the Swash Channel by 0.09 miles per hour.

"In the spring of 1872 Brigadier-General (then Lieutenant-Colonel) John Newton, Corps of Engineers, had a series of current observations taken in the lower bay, with the following results:

PLACE.	Surface velocity in miles per hour.	Bottom velocity in miles per hour.
Fourteen-Foot Channel..	2.20	0.40
East Channel...	1.65	0.90
Swash Channel..	2.00	0.40
Main Channel, close to and north of the point of Sandy Hook..	2.40	0.80

"These observations were stated to have been taken at the maximum velocity of the ebb tide."

From these tables it is seen that the flow is more rapid in the Fourteen-Foot than in the Swash Channel, and 100 per cent. greater in the Swash than in the Main Ship Channel; showing the tendency of the water to take the course of the shortest distance from the upper part of the Main Ship Channel to the ocean.

The question of improving the Swash Channel does not imply the substitution of the Swash for the Main Ship Channel, for were the

former channel improved and employed, both ends of the latter channel would still be used in conjunction with the Swash Channel. Considering the Main Ship and the Gedney's channels, as now improved, to be one and the same channel extending from the Narrows to deep water of the ocean, it will be seen that while its course from the Narrows to Flynn's Knoll is direct, and not seriously indirect from Flynn's Knoll to the ocean, yet its course from the Narrows to the ocean is very disadvantageously indirect.

By referring to the map, it will be seen, as Colonel McFarland says in his report for 1886, that:

"The Swash Channel is really a cut-off from the Main Ship Channel, leaving it about 6 miles below the Narrows, and joining it again at the eastern end of Gedney's Channel. The distance from the Narrows to 30-foot soundings outside the bar by the Main Ship Channel is 15 miles and by the Swash Channel 11 miles."

Hence from the Narrows to the ocean the distance is 4 miles less by way of the Swash than by the Main Ship Channel. The directions of the Beacon ranges of these two channels, according to the Government chart, shows that the deviation of the Swash Channel from the course of the Main Ship Channel, at their upper union, is 54 degrees; and at their lower union the deviation of the Swash from the course of the Main Ship Channel is 61 degrees; the sum of these two angles being 115 degrees; while the sum of two angles formed by the Beacon ranges in their course around Flynn's Knoll is also 115 degrees, being 50 degrees and 65 degrees, respectively. Hence from the Narrows to the inner mouth of the Gedney's Channel, the amount of deviation of the Beacon ranges from a straight course is the same, and the number of angles or turns is the same, by way of both channels. And there is not a great difference between the magnitude of the several angles, the greatest being 65 degrees and the least being 50 degrees, both in the Main Ship Channel. Colonel McFarland, referring to these angles in his report for 1886, says:

"At the lower end of the Swash Channel it is necessary to make quite a sharp turn to the northward in order to get into the Gedney's Channel, and the turn from the Main Ship Channel into the Gedney's Channel is even shorter. More room should be given for vessels at these points."

Considering the extent of ocean traffic and travel and its rapid increase, it would seem, for convenience and safety, that, that portion of

the Main Ship Channel which turns upon itself to the extent of 115 degrees around Flynn's Knoll, should be relieved of a portion, if not all, of the larger ocean vessels ; not only because of the greater distance, inconvenience, expense, loss of time and danger attendant upon its navigation, but because of its less capacity than would be that of the improved Swash Channel. The great length and draught of many steamers require so large a turning area, that in doubling Flynn's Knoll or in following any short turns of the channels they are liable to swerve from one side of the channel to the other, causing increased liability of collision and grounding, as well as diminishing the capacity of the channel. In short, the relative safety and capacity of and the convenience and speed of passage through a channel of given width and depth, are commensurate with its straightness and inversely with its length. Besides, the current in the Swash Channel runs in line with its axis, while in that portion of the Main Ship Channel lying north and south the current is transverse to the axis of the channels, which diminishes the facility of its navigation. Again, as a great majority of the vessels entering and leaving the port pass through the Swash Channel, and as in passing out of this channel at either end their course is in a direction diagonal to the axis of the Main Ship Channel, and therefore to the course of the larger ocean steamers in their passage through the latter channel, it is evident that this is a source of liability of collision that would be obviated if all vessels were running in the same general direction.

Though the items of extra fuel and time, required by the indirection of the source of and the greater distance by, the Main Ship Channel, may be considered of small account, nevertheless, in view of the increasing number of large ocean steamers navigating these channels and the interest felt in saving time, especially with that class of steamers that are now obliged to employ the Main Ship Channel, they are not to be overlooked. The saving on these two items of fuel and time, to say nothing of other advantages, would aggregate a sum, in course of time, sufficient to pay the cost of amplifying the Swash Channel to a width of 1 000 feet and a depth of 30 feet at mean low water. An evidence of the advantages of the Swash over the Main Ship Channel is the fact that it is employed by nearly all, if not all, navigators who are not compelled to take the Main Ship Channel because of great draught of their vessels.

The Main Ship Channel, being a trifle deeper as well as somewhat wider, than the Swash Channel, it has been employed by the larger

steamers, and the use of it by these deep draught vessels has had the effect of keeping it open and increasing its depth. But as the current in the Swash is twice as rapid as in the Main Ship Channel, and in line with instead of across its axis, had the bed of the Swash been disturbed and stirred up in the same manner, it is possible that the depth of the Swash would now be equal to that of the Main Ship Channel before its recent improvement.

In his report for 1885 Colonel Gillespie says:

"The change in the character and draught of the vessels using the harbor during the past thirty years, particularly in the last ten years, has had its influence, no doubt, in slightly increasing the depth from 21 to 24 feet on the bar."

Why the Main Ship Channel between the upper and lower ends of the Swash, instead of the Swash Channel itself, was improved, may have been, presumably, for the reasons that: At the time the Government concluded to improve New York Harbor its engineers were not clear as to the best project to present or the best course to pursue, not knowing without trial what would be the result of dredging as to permanency; therefore they felt their way, step by step. Had they known at the outset that the entire work could be done by dredging alone, and had they then known, as they now do, that the results of dredging would be successful and permanent, possibly they might have concluded that the Swash was the preferable channel to improve first.

The project of improving this channel can now be viewed in the light of facts and experience which did not previously exist.

DESCRIPTION OF PLATES.

CIII. Chart showing the channels in the Lower Bay.

CIV. View of the dredging steamer *Reliance*.

CV. "Sheer-plan" of the after part of the *Reliance*, showing the arrangement of the drags and suction pipes.

CVI. Plan of main deck of after part of the *Reliance*, showing the arrangement of the boilers, propelling engines, main pumping engines, dredging pumps and auxiliary machinery.

CVII. Sections of the steamer between propeller engines and boilers, looking forward and aft.

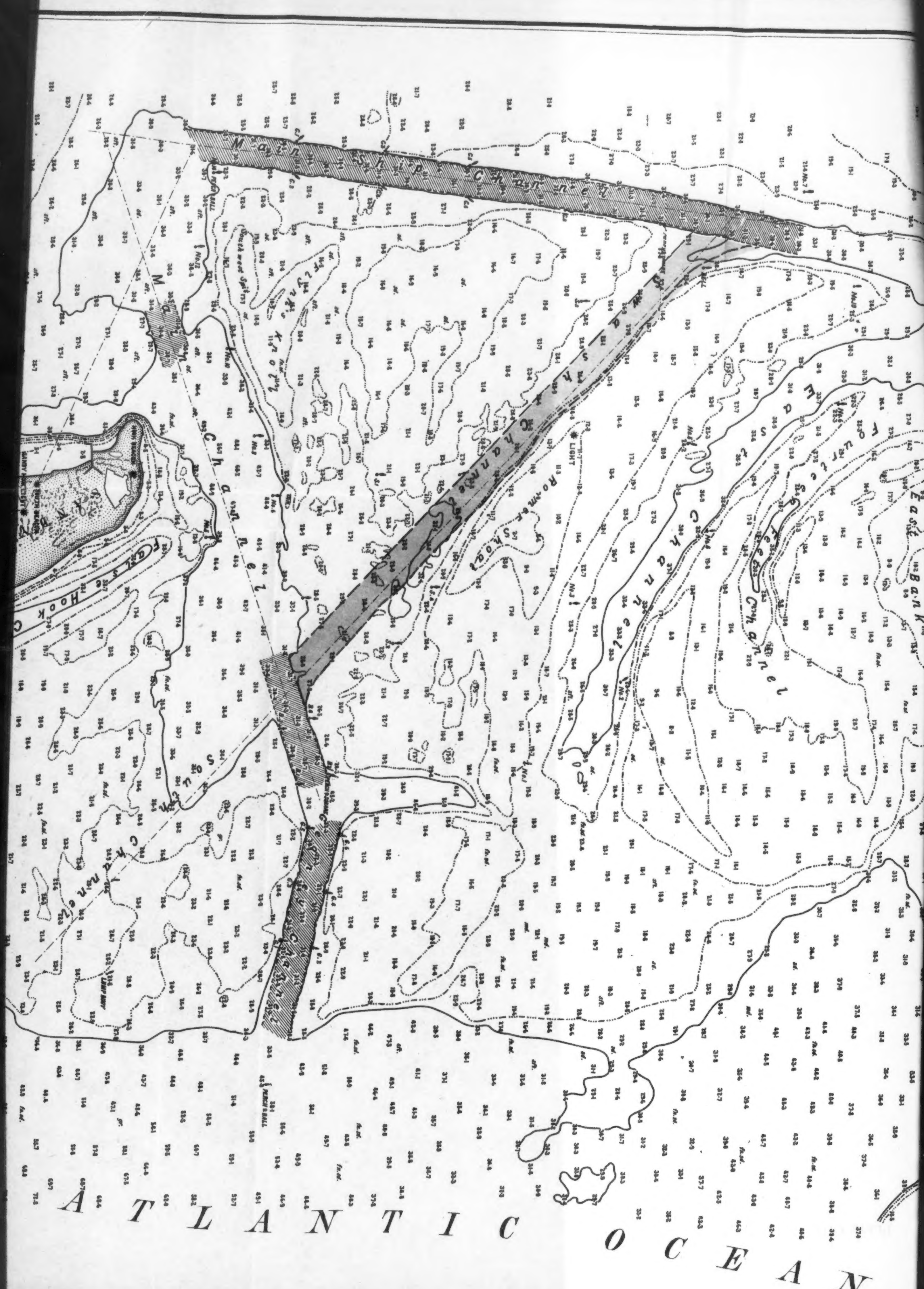
CVIII. Details of the drags.

CIX. General plan of a dump scow.

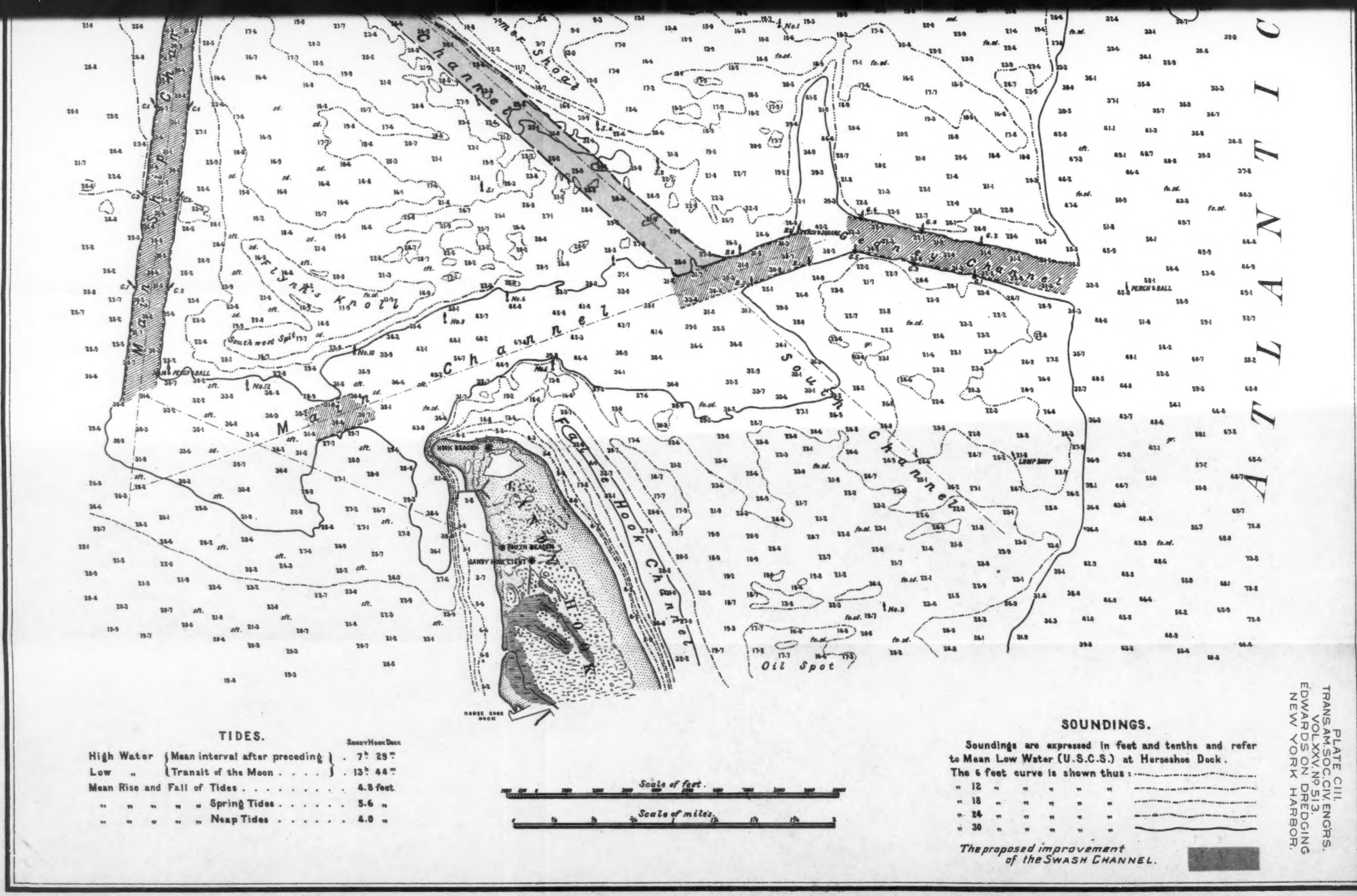
CX. Longitudinal section of the after part of the *Reliance*, showing machinery, pumps, etc., and one of the bins for dredged material.

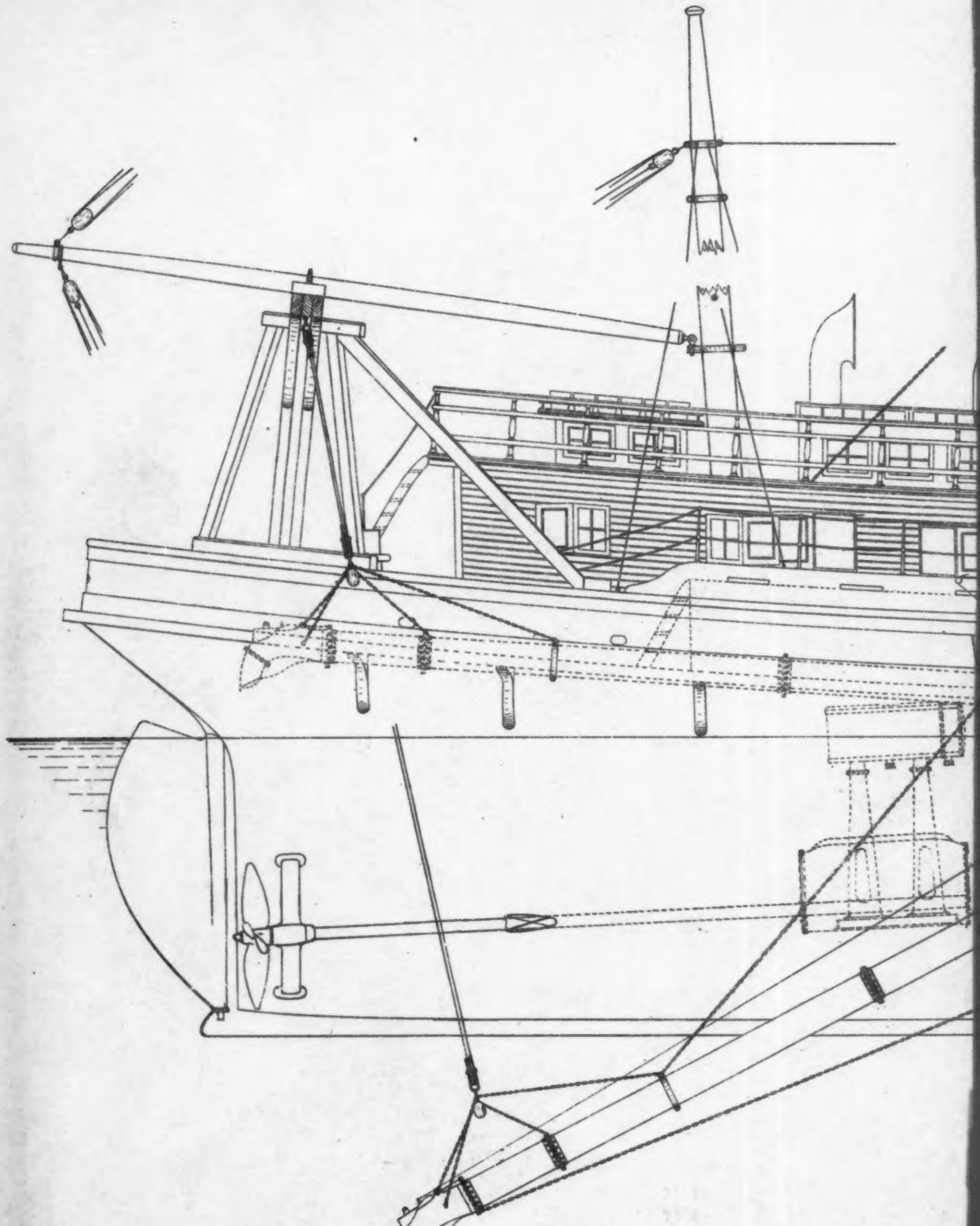
This paper, for special reasons, is printed without time being given for discussion, and discussions may be sent in for printing at a later date.

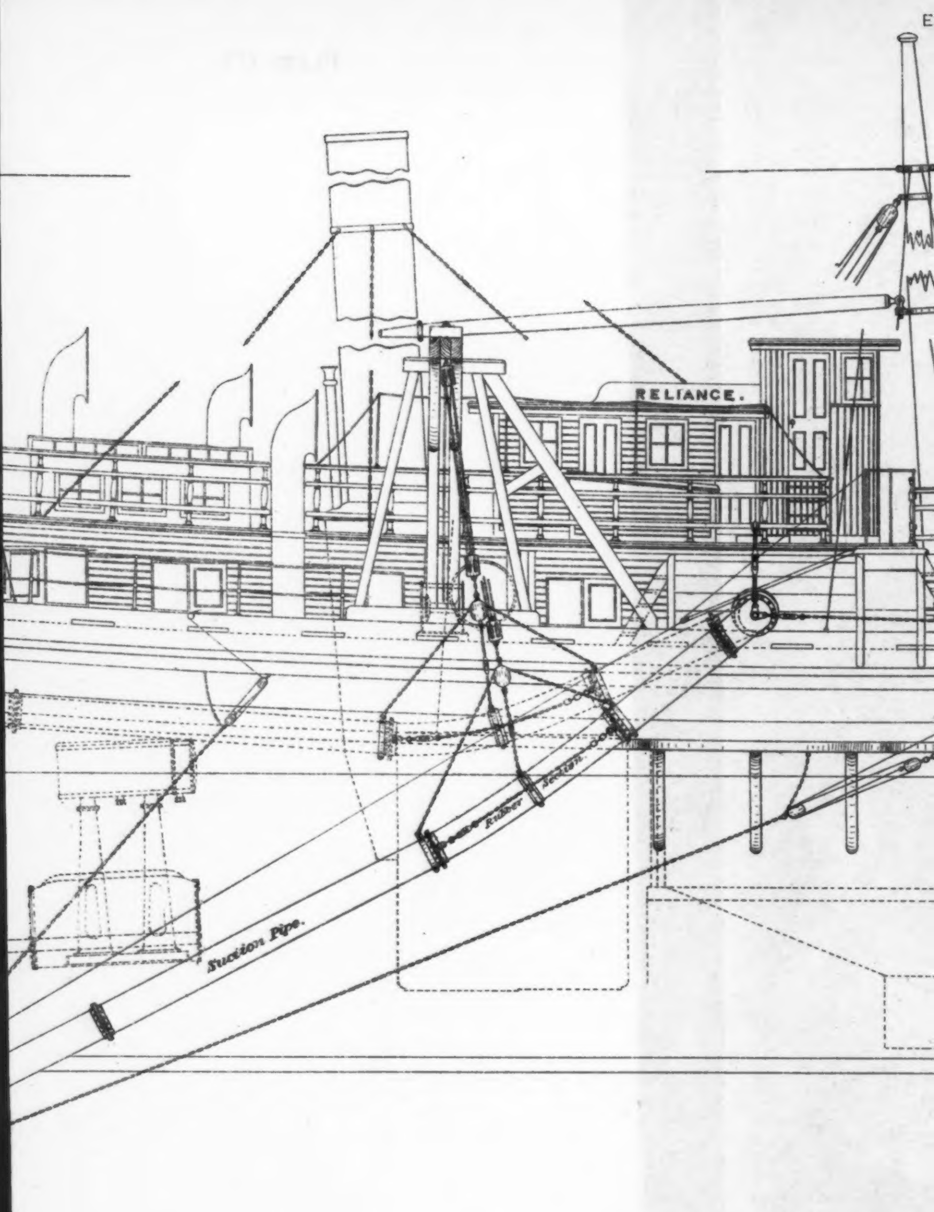

IMPROVEMENT OF
NEW YORK HARBOR, NEW YORK,
IN CHARGE OF
LIEUT. COL. G. L. GILLESPIE,
CORPS OF ENGINEERS, U.S.A.
MAP OF THE
LOWER BAY
Showing
the improved Channels dredged to 30 feet, mean low water,
completed October 1891.
SCALE 1/30,000
ENGINEER OFFICE, U.S. ARMY.
New York, N.Y., Dec. 2, 1891.
This Map is compiled from the U.S. Engineer survey of 1884, supplemented by additional surveys of the improved channels made in June 1891
G. L. Gillespie
Lieut. Col. of Engineers, U.S.A.
HOFFMAN ISLAND
SWINBURN HOSPITAL ISL.
West Bank
CONEY ISLAND
SHEEPSHEAD BAY
Rockaway Inlet
East Bank
Fourteen



Main Ship Channel
Swash Channel
East Bank
Fourteen Feet Channel
East Channel
LIGHT
Romer Shoal
Flynn's Knoll
South west Spit
Main Channel
South Channel
Gedney Channel
False Hook Channel
LUMP BUOY
PERCH & BALL
ATLANTIC OCEAN


ATLANTIC
Main Ship Channel
Main Channel
Gedney Channel
South Channel
False Hook Channel
Flynn's Knoll
South west Spit
Sandy Hook
Oil Spot
High Beacon
South Beacon
Sandy Hook Light
Horse Shoe Dock
Perch & Ball
Lump Buoy
TIDES.
Sandy Hook Dock
High Water { Mean interval after preceding } . 7h 29m
Low " { Transit of the Moon . . . . } . 13h 44m
Mean Rise and Fall of Tides . . . . 4.8 feet.
" " " " " Spring Tides . . . . . . 5.6 "
" " " " " Neap Tides . . . . . . 4.0 "
Scale of feet.
Scale of miles.
SOUNDINGS.
Soundings are expressed in feet and tenths and refer to Mean Low Water (U.S.C.S.) at Horseshoe Dock.
The 6 feet curve is shown thus :
" 12 " " " " "
" 18 " " " " "
" 24 " " " " "
" 30 " " " " "
The proposed improvement of the SWASH CHANNEL.
PLATE CIII.
TRANS.AM.SOC.CIV.ENGRS.
VOL.XXV. No 513.
EDWARDS ON DREDGING
NEW YORK HARBOR.





RELIANCE.
Suction Pipe.
Rubber Section.

PLATE CV
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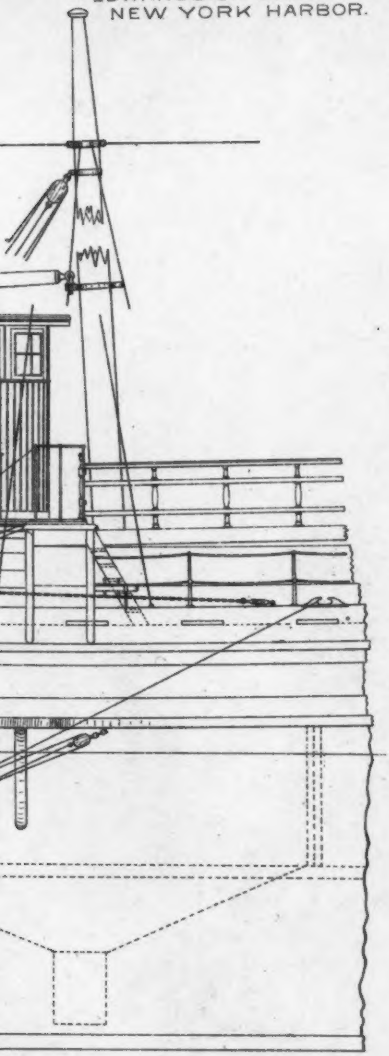

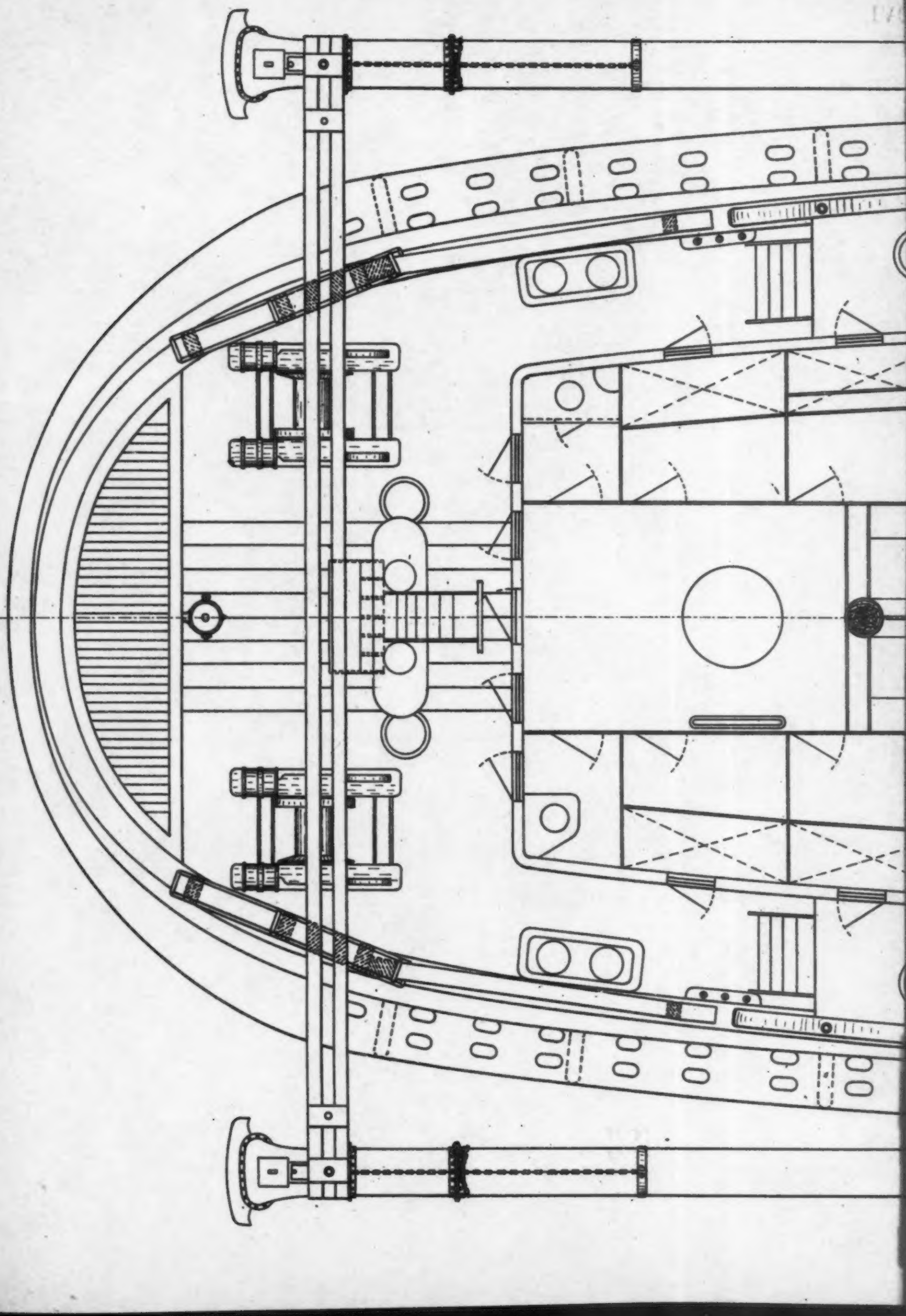

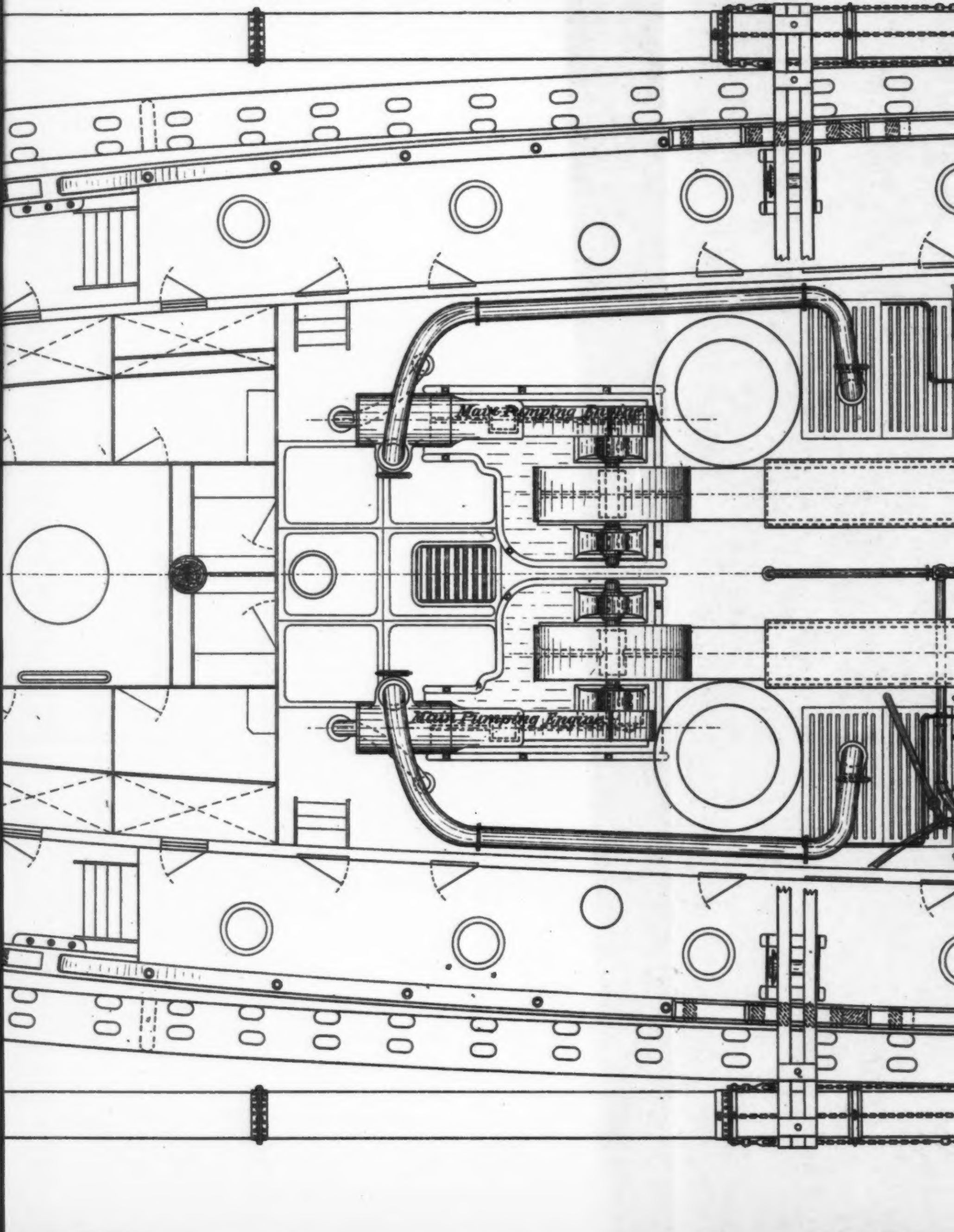
Main Pumping Engine
Main Pumping Engine

PLATE CVI
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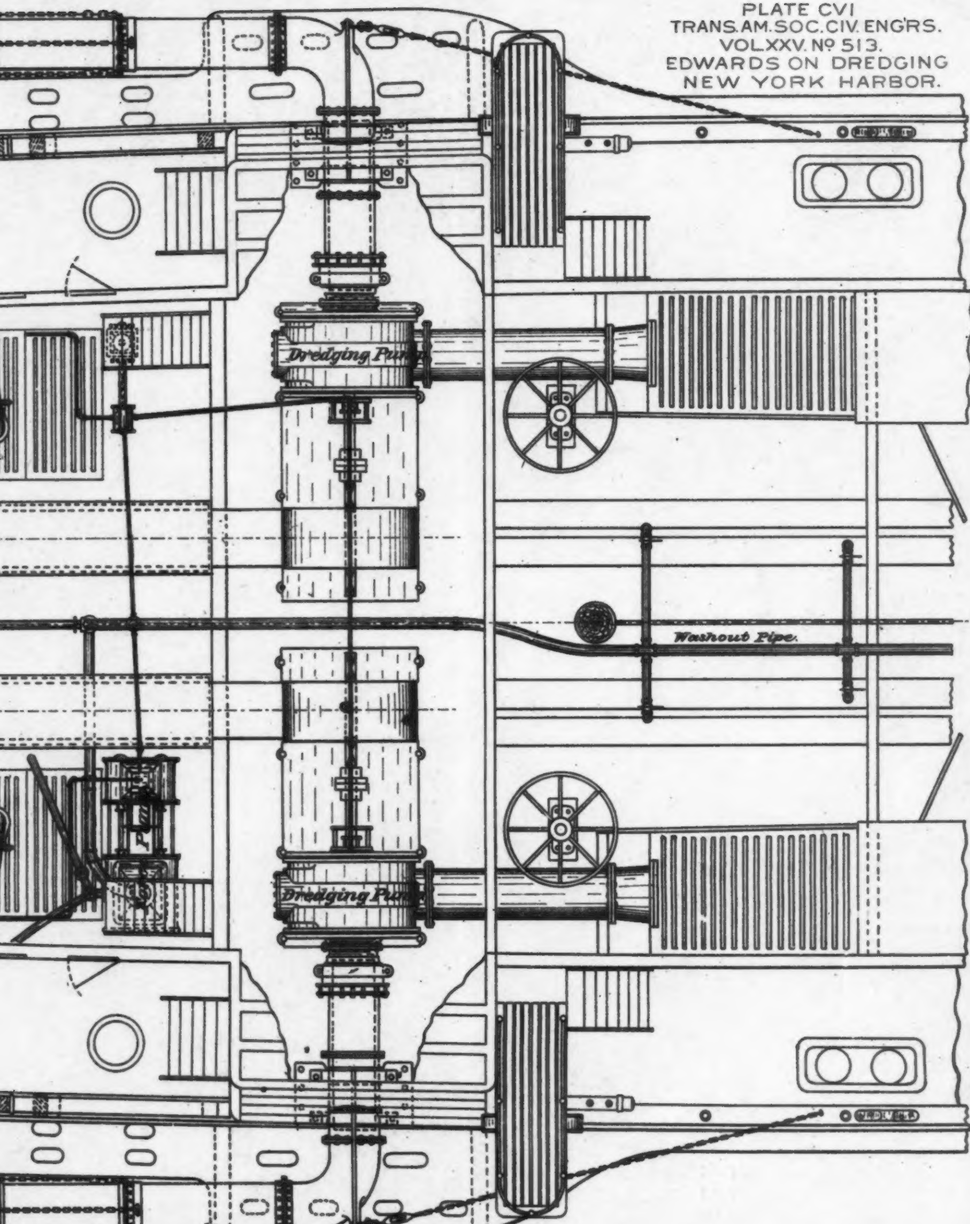

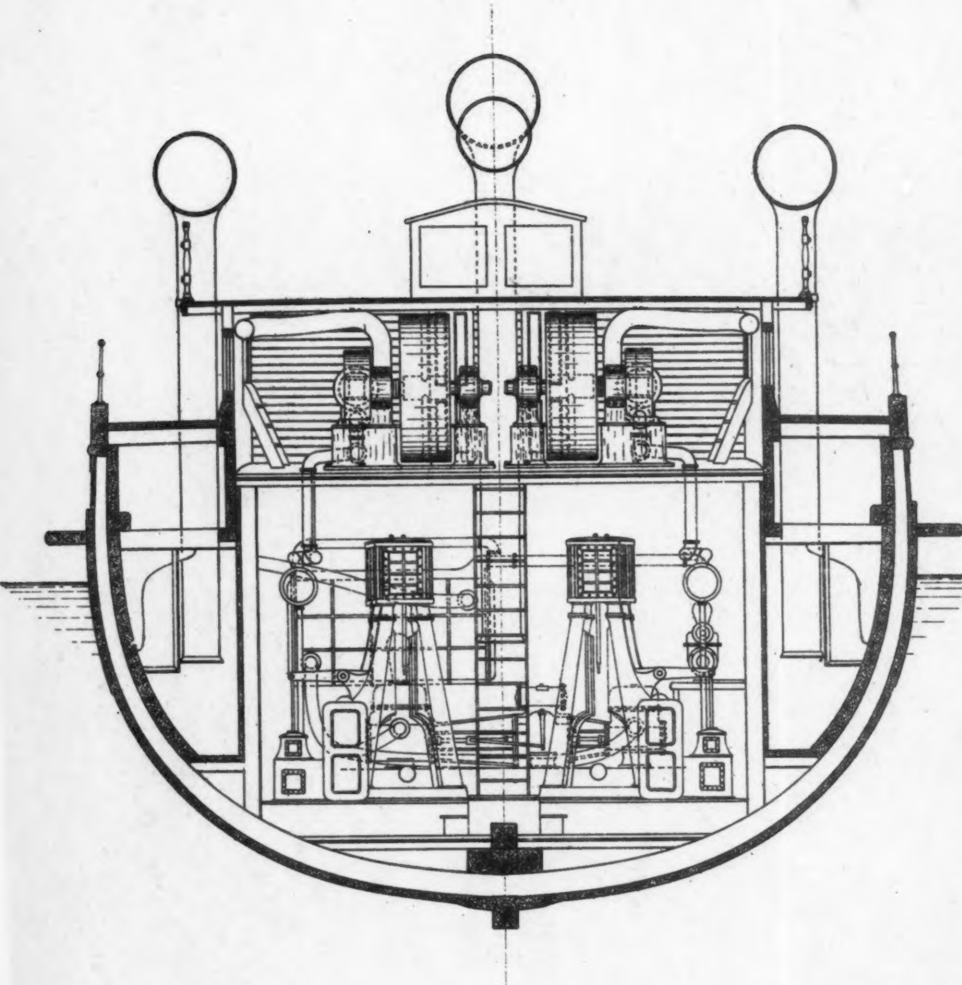

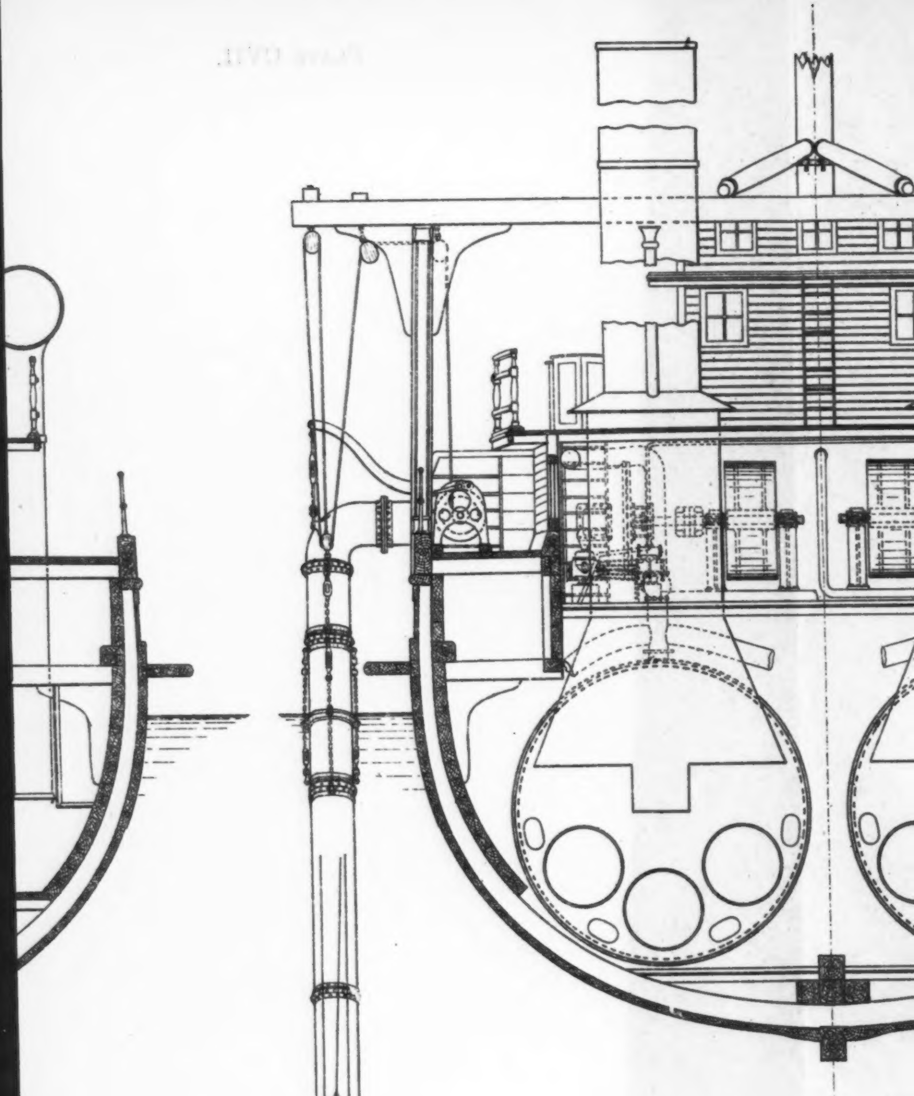

PLATE CVII
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EDWARDS ON DREDGING
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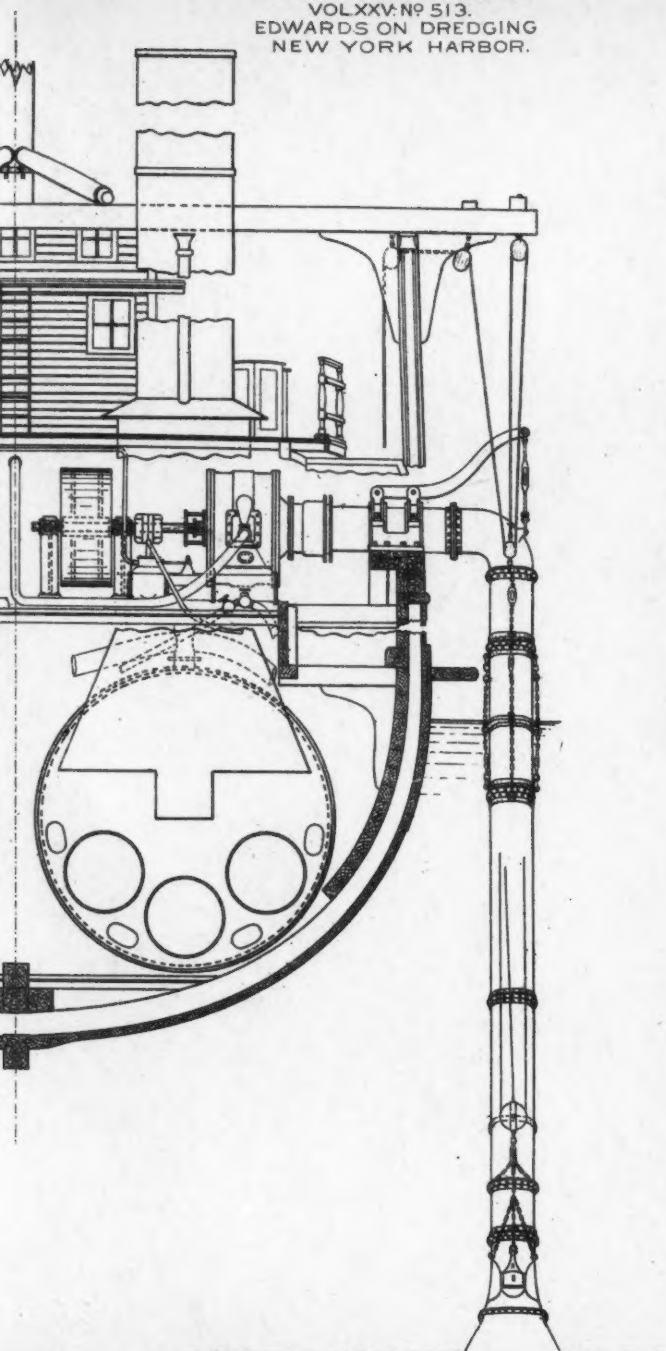

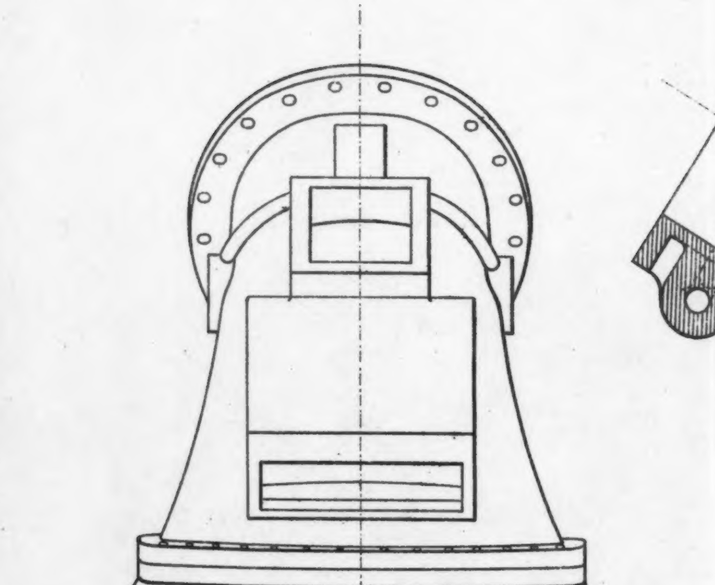

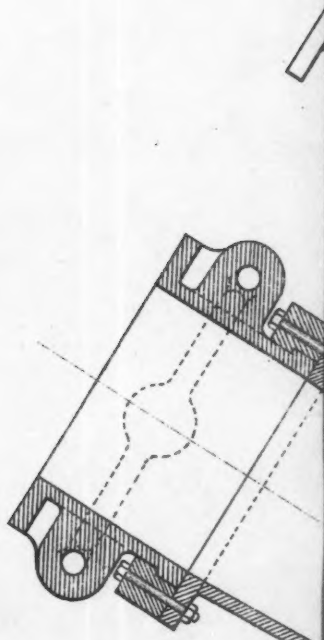

PLATE CVIII
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VOL. XXV. № 513.
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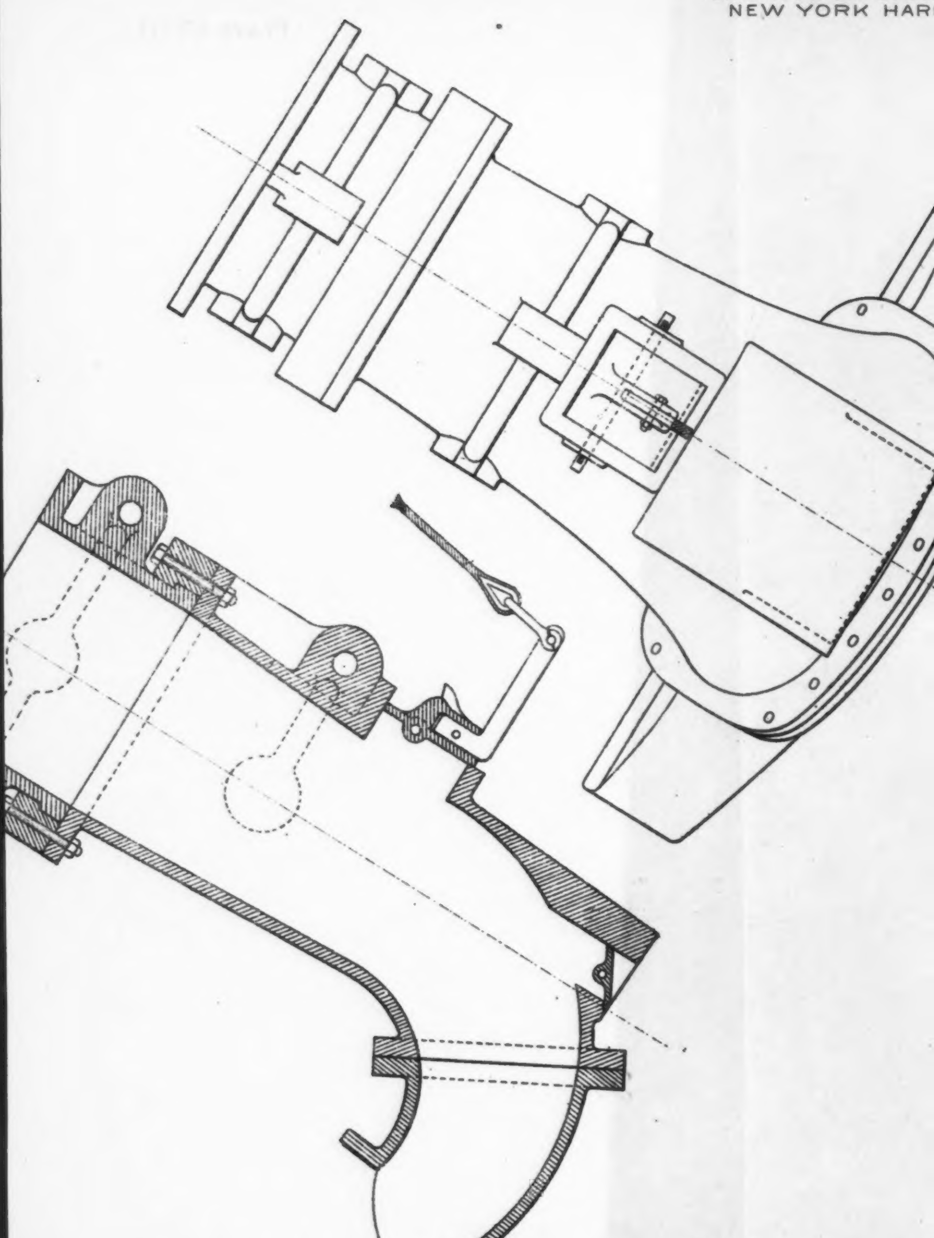
NEW YORK HAR



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XV. Nº 513.
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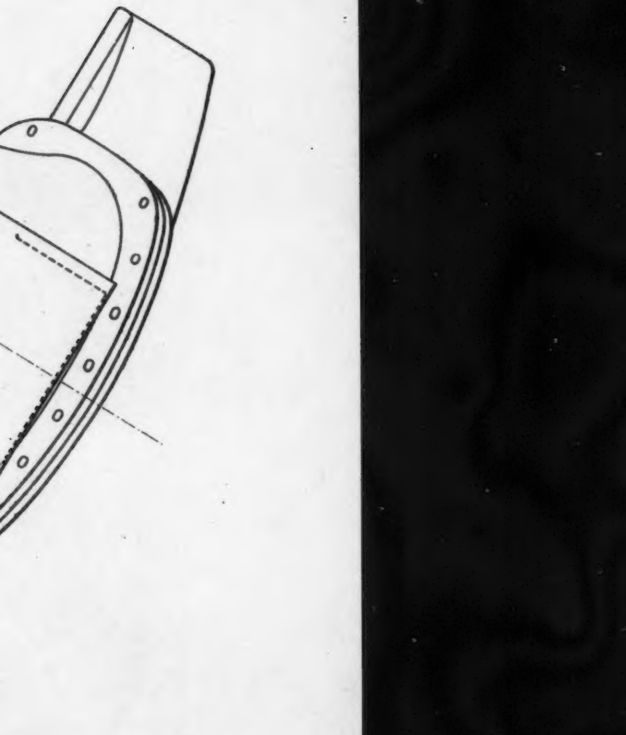

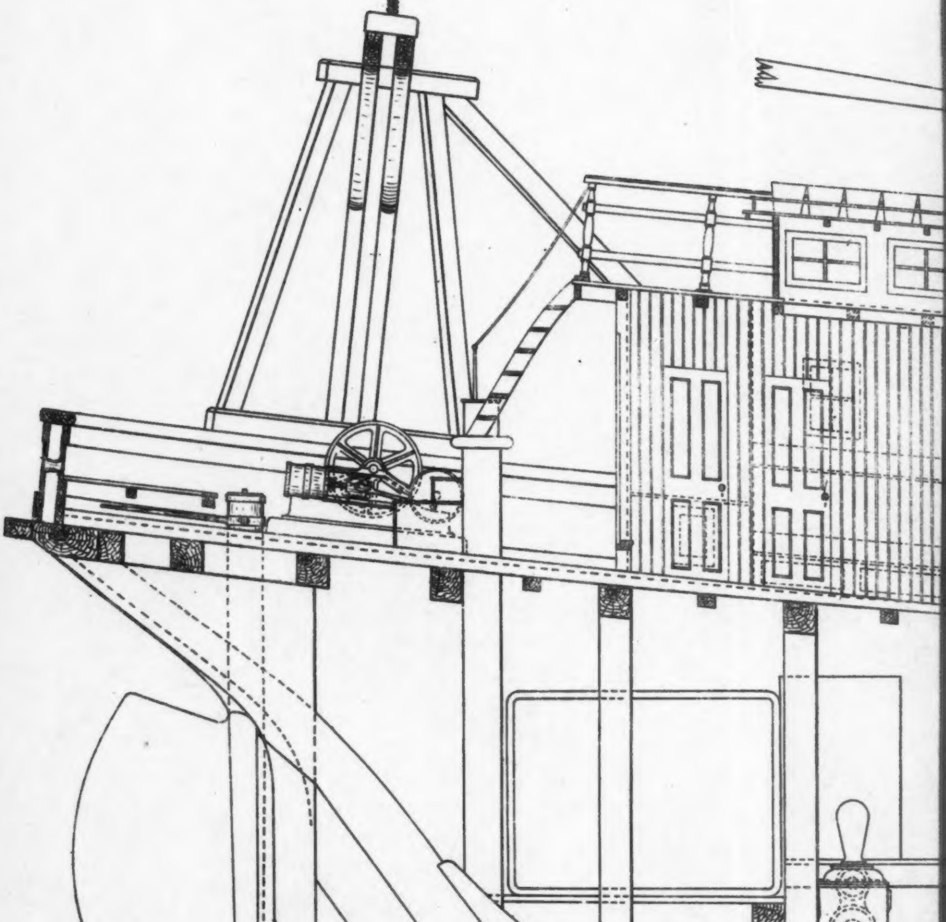

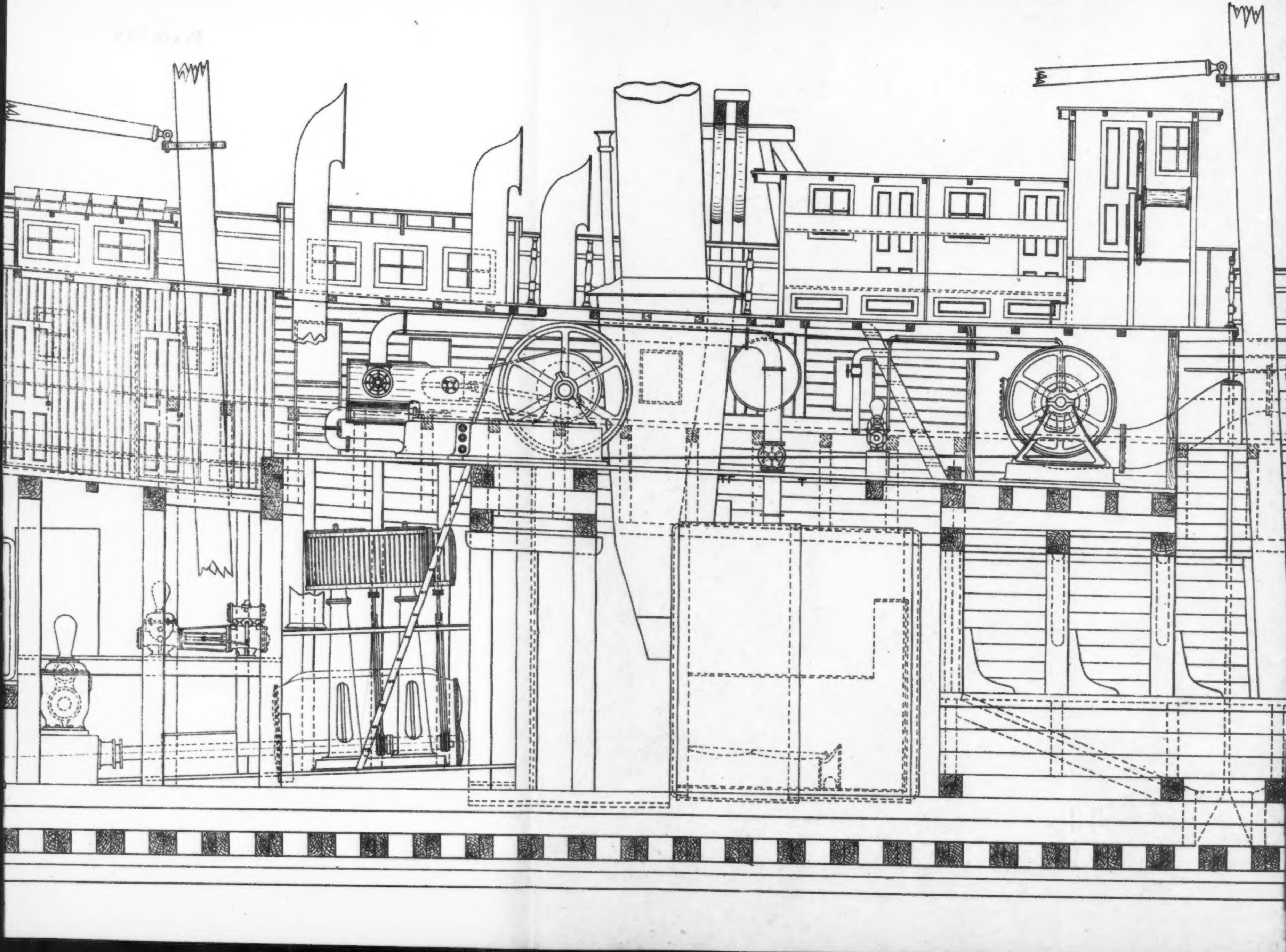

PLATE CIX
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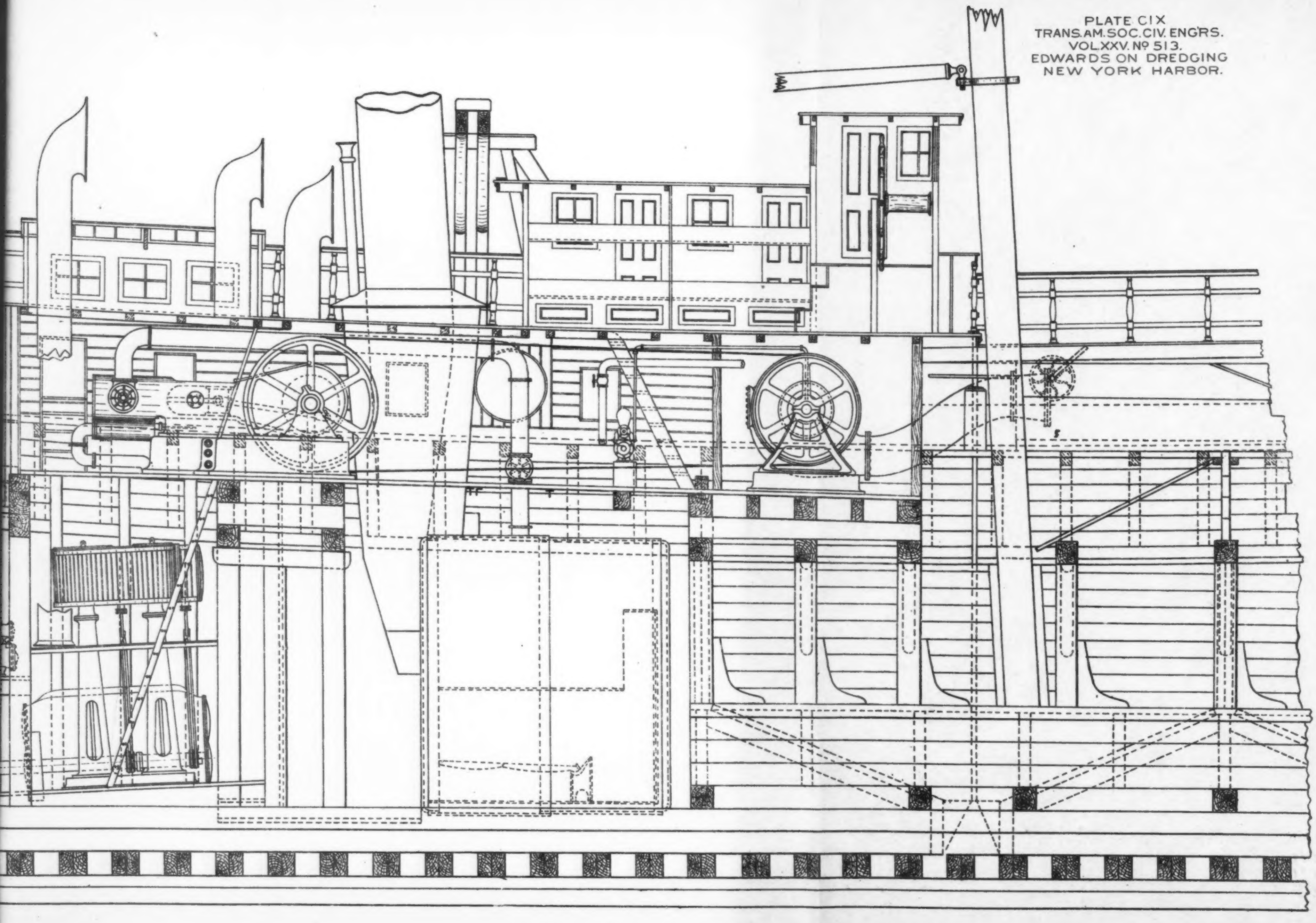

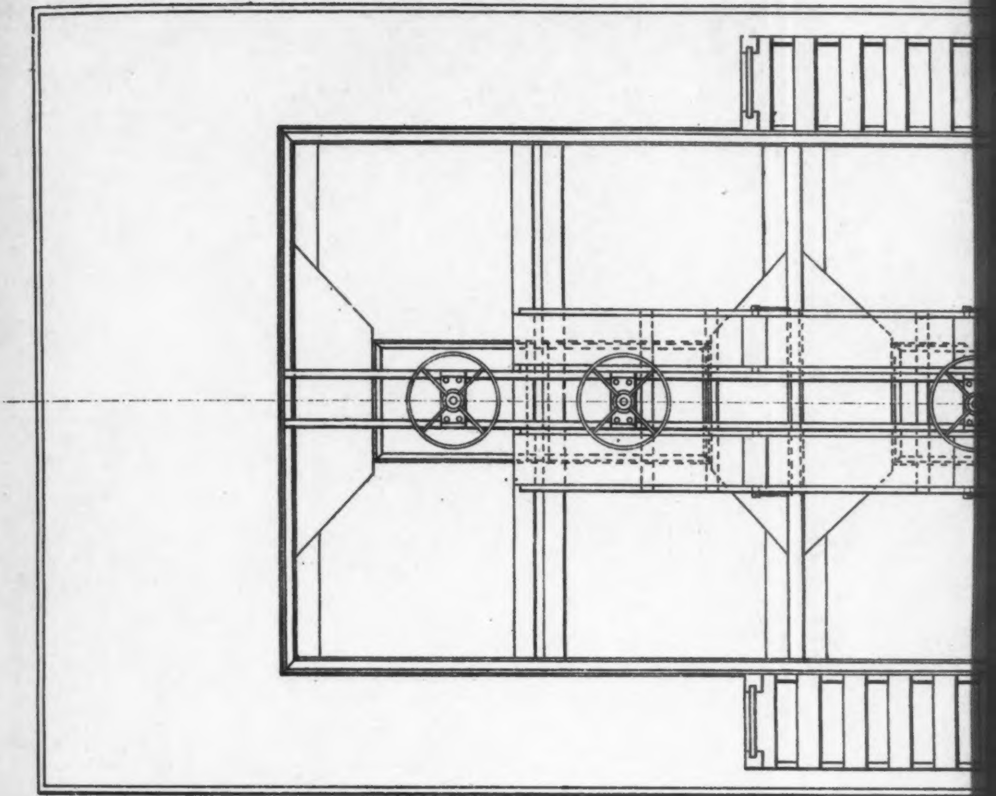

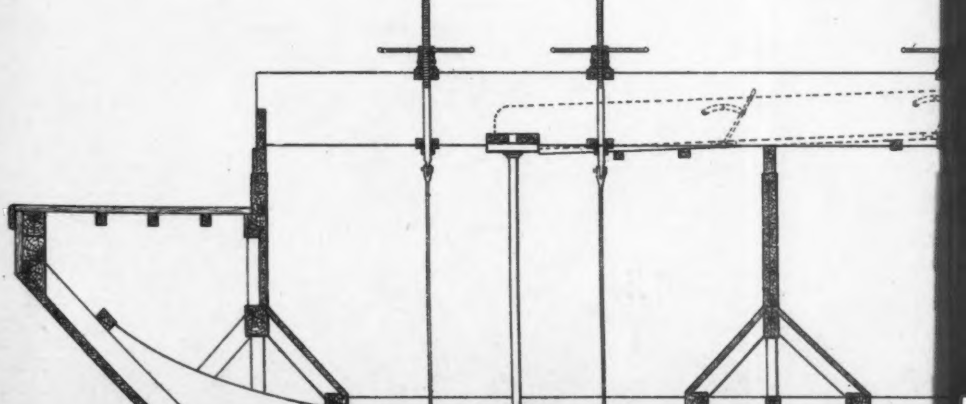

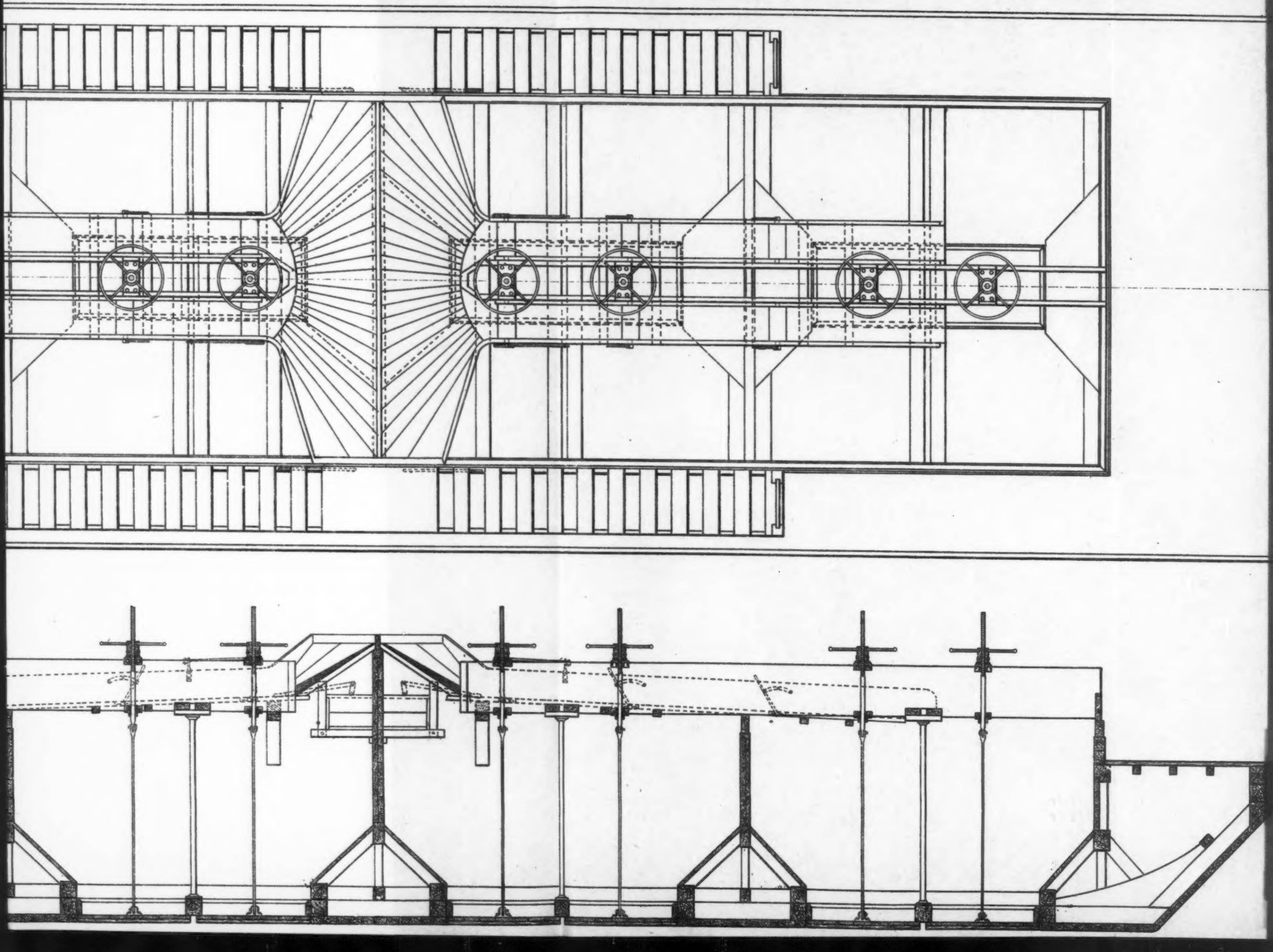

PLATE CX
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